Engineering Properties and Mineralogical Composition of Expansive Clays in Al-Qatif Area (K.S.A.)

by

Rafi Ahmad

A Thesis Presented to the

FACULTY OF THE COLLEGE OF GRADUATE STUDIES

KING FAHD UNIVERSITY OF PETROLEUM & MINERALS

DHAHRAN, SAUDI ARABIA

In Partial Fulfillment of the
Requirements for the Degree of

MASTER OF SCIENCE

In

CIVIL ENGINEERING

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This thesis, written by

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under the direction of his thesis committee, and approved by all the members, has been presented to and accepted by the Dean, College of Graduate Studies, in partial fulfillment of the requirements for the degree of

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Dedicated to

My Newly Born Son Malik, the Farex Baby

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ABSTRACT

Al-Qatif is a town in the Eastern Province of Saudi Arabia where expansive soil problems are reported. This study is concerned with the determination of engineering properties and behavior of these clays by performing various laboratory tests.

A site investigation was carried out to obtain undisturbed block samples. Geotechnical properties e.g. Atterberg limits, natural water content and density, showed that Al-Qatif clays are highly expansive and are heterogeneous in nature.

Quantitative characterization to estimate the amount of anticipated volume change was carried out using conventional one-dimensional consolidometer. The results of direct swelling tests indicated high values of swelling pressure and percentage of swell.

X-ray diffraction and scanning electron microscopic analysis indicated that Al-Qatif clays are mixtures of two principal clay minerals: montmorillonite (pure smectite) and polygorskite with flaky flocculated morphology & needle form. Chemical analysis showed the presence of low amount of organic matter. All soil samples were alkaline in character with high cec values.

Laboratory evaluation of chemical stabilization was done using three types of chemicals. The results showed that 4-8% of commercial lime should be preferred for preconstruction treatment of Al-Qatif clays. Some methods of special foundation designs and floor slab construction are also suggested.

الخلاصة

تقع مدينة القطيف فى المنطقة الشرقية من المملكة العربية السعودية حيث تم التبليغ عن وجود مشاكل متعلقه بالتربه المنتفخة . وتهتم هذه الدراسة بايجاد الخواص الهندسية وتصرف هذه التربة الطينية باجراء العديد من الأختبارات المعملية .

وقد اجرى استقصاء ميدانى للحصول على عينات غير مفككه . واوضحت الخواص الجيوتكنيكية التى تتضمن حدود اتربرج والمحتوى المائى الطبيعى والكثافة أن تربه القطيف شديدة الأنتفاخ ومتغايره الخواص .

هذا وقد قدر تغير الحجم المتوقع باستخدام مقياس الاندماجية التقليدى ذو الاتجاه الواحد وبينت نتائج اختبارات الانتفاخ المباشره عن قيم عاليه لضغط ونسبة الانتفاخ .

كذلك فقد اظهرت حيود الأشعة السينيه والميكروسكوب الألكترونى أن تربة القطيف الطينية تتكون غالبا من خليط معدنى المونتموريللونيت (سمكتيت) والباليجورسكيت ذو الصفائح الدقيقة المبعثره وبعض الأنسجه الأبريه الشكل ، واوضح التحليل الكيمائى عن وجود نسبة ضيئلة من المواد العضوية . وكانت عينات التربة جميعها قلوية مع قدرة عالية لأستبدال الأيونات الموجبه الشحنه (كاتيون) .

ولقد تم تقييم التثبيت الكيمائى باستخدام ثلاثة أنواع من المواد الكيماوية . وبينت النتائج عن أفضلية استخدام نسبة تتراوح بين ٤و٨ فى المائه من الجير التجارى فى معالجة تربة القطيف الطينية . واقترحت بعض الطرق الخاصة لتصميم القواعد والبلاطات الأرضية التى ستنفذ على هذه التربة .

* * *

CHAPTER 1

INTRODUCTION

1.1 General

Problems associated with expansive clay soils are worldwide. Having occured in countries like Africa, Australia, India, Saudi Arabia, South America, Canada and the United States. Such soils have caused great damage to structures. The problems with foundations on expansive soils include heaving, cracking and breaking of pavements, building foundations and slab-on-grade members. The cost of these damages in the United States alone amounts to some $2.3 billion annually - more than twice the damage from floods, hurricanes, tornadoes and earthquakes [8].

Until 1930 it was unknown to Soil Engineers that expansive soils cause problems. Prior to 1920 most of the lightly loaded buildings in the United States consisted of frame dwellings. Such structures could withstand considerable movements without exhibiting noticeable cracks. By 1930 brick construction became widely used. Then the owner found cracks developing in the brick course But at that time the damages were attributed to inferior construction and settlement of foundation at one corner, without recognition of the role of expansive soils. The U.S. Bureau of reclamation first recognised the swelling soil problem in 1938 in connection with a foundation for

steel siphon in Oregon. Since that time Soil Engineers realized the cause of damage was sometimes other than settlement. The increasingly extensive use of concrete slab-on-ground construction after 1940, has further increased the damage to structures caused by swelling soils. In the last 20 years soil engineers turned their attention to this very complicated problem. Although much research and study have been directed to swelling soils, the basic problem of how to deal with lightly loaded structures on swelling soils remains unsolved. A practical method of how to design a structure that will be safe against heave within economical reach has yet to be found [7].

Distribution of swelling soils is generally a result of geologic history, sedimentation and local climatic conditions. Arid climatic conditions and severe weathering environment prevailing in the Arabian Peninsula promote the widespread occurrence of expansive soils in this region. In Saudi Arabia, specially in its southern and eastern parts, expansive soils are derived from argillaceous rocks of Permian, Cretaceous and Tertiary age. Here in the Kingdom, recent development has extended to areas the soils of which have relatively high swell potential. It is believed that numerous other places possessing expansive soils also exist, which are not yet located [5].

1.2 Problem definition

Al-Qatif area lies on the western coast of the Arabian Gulf, in the Eastern Province of Saudi Arabia. This town is about 20 kms north of Dammam, the main port city of the Kingdom. It is bounded

approximately by the latitudes 26° 35'N to 26° 40'N and the longitudes 49° 34'E to 49° 37'E. In this region the upper strata is of Miocene age and consist of terrestrial sediments. These deposits, known as Hadruk Formation, consist of greyish green clays of middle Miocene age, which have been identified as expansive in nature [4]. Heaving of sidewalk & floor slabs, wall cracks and breakup of pavements are common phenomenon in this area, Figs. 1.1 & 1.2. Until now not much is known about the nature and behavior of these swelling soils. In this study, expansive soils of Al-Qatif area are being studied in detail. More emphasis is given to predict the engineering properties and the swelling characteristics of this soil. Since the type of clay minerals plays an important role in the determination of heave in expansive soils, efforts are being made to identify those minerals both qualitatively and quantitatively using latest technologies like X-ray diffraction and Scanning Electron Microscopy (SEM).

1.3 Objectives of the Study

The overall objective of the study is to determine the properties and behavior of Al-Qatif clays by performing different laboratory tests and to come up with some stabilization methods to overcome heave problem. More specifically, the objectives are;

1. Identification, classification and distinguishing features of expansive clays in Al-Qatif area.
2. Prediction of swelling pressure and swelling potential by performing direct laboratory tests.

Fig. 1.1(a) : Dessicated Cracks in Al-Qatif Formation

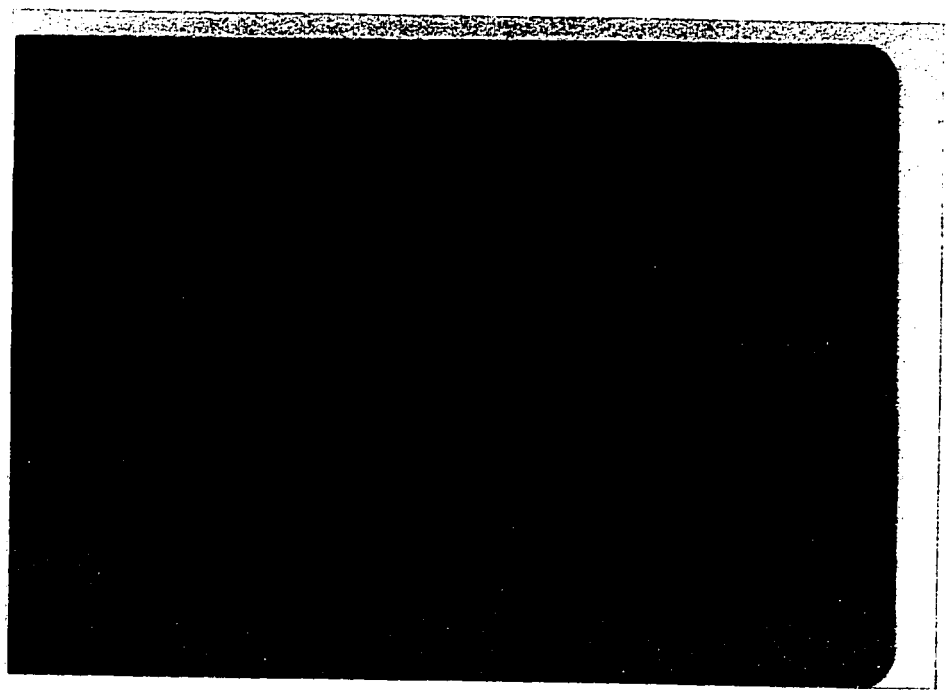


Fig. 1.1(b) : Upheaving of Floor Slab in Al-Qatif Area

Fig. 1.1 : Expansive Soil Problems at Al-Qatif Housing Project Site

Fig. 1.2(a) : Progressive Cracks in Boundary Wall

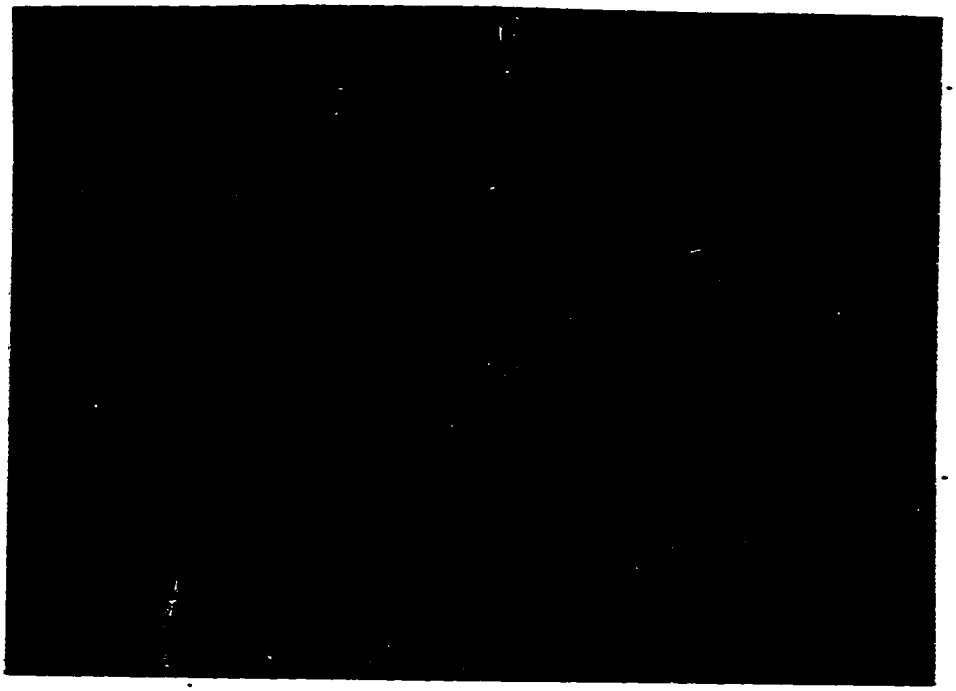


Fig. 1.2(b) : Wall Cracks & Heaving of Sidewalk

Fig. 1.2 : Expansive Soil Problems in Old Anak Town, Al-Qatif

CHAPTER 2

LITERATURE REVIEW

2.1 Distribution of Expansive Soils

2.1.1 Around the World

Expansive soils are distributed all over the world. The countries in which expansive soils have been reported are as shown in Fig. 2.1. Potentially expansive soils can be found almost anywhere in the world. In the underdeveloped nations, much of the expansive soil problem may not have been recognized. It is to be expected that more expansive soil regions will be discovered each year as the amount of construction increases [7].

Fig. 2.1 shows that the potentially expansive soils are confined to the arid and semi-arid regions where the evapotranspiration exceeds the precipitation. This follows the theory that in arid and semi-arid zones, the lack of leaching has been the cause of formation of montmorillonite.

2.1.2 In the United States of America

Expansive clays are found throughout the United States. If allowed to expand freely in the field, some have the potential to expand 30% to as much as 1,000% [1]. Most sites of structures where instances of heaving are reported, contain argillaceous soils. Since

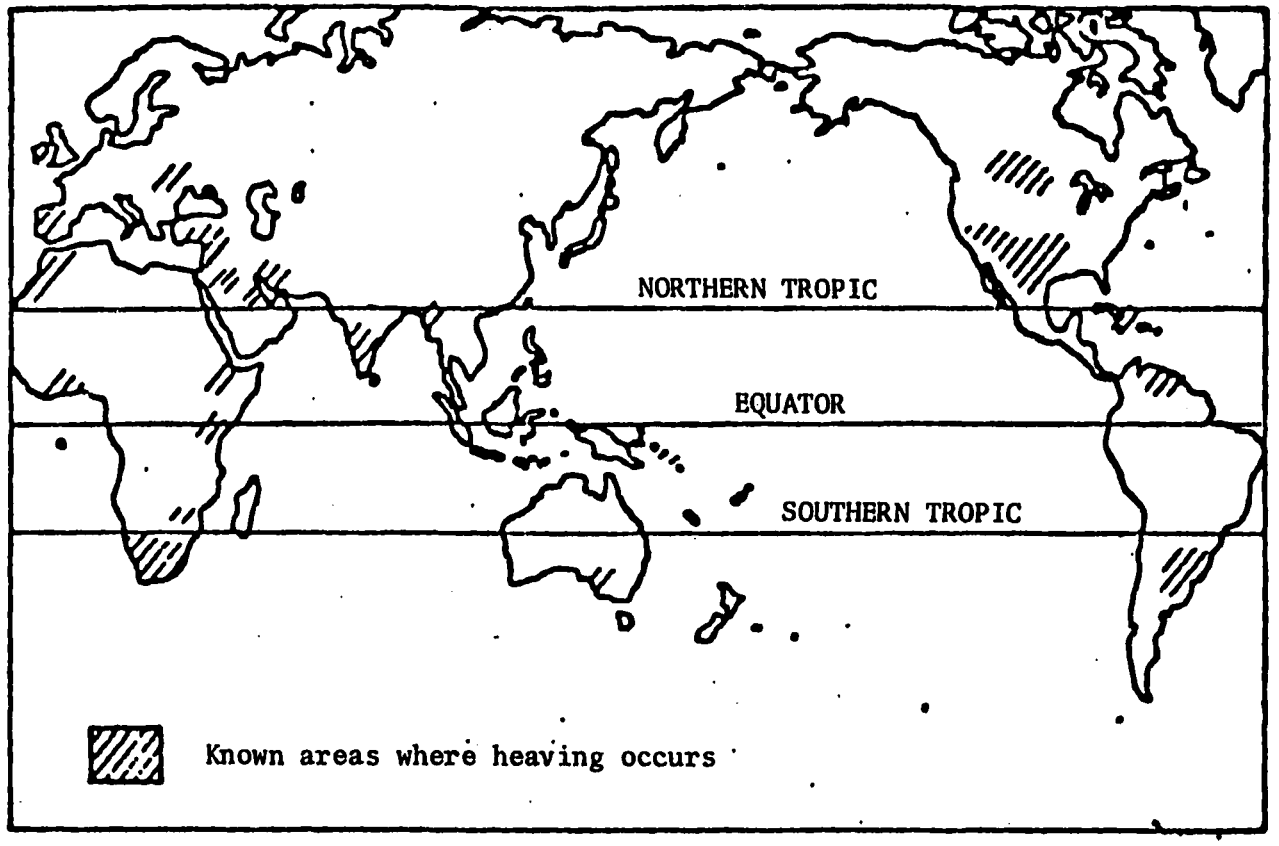


Fig. 2.1 : Distribution of Expansive Soils Around the World
(Donaldson, 1969; Ref. 2, Modified)

most of these structures are hydraulic, the normal difficulties encountered when expansive clays are present in the subgrade foundation are greatly magnified. As a result of some of the difficulties that have been experienced, considerable research has been undertaken so that such type of soils can be recognized and their potential swelling properties anticipated.

In the United States, from the Gulf of Mexico to the Canadian Border, and from Nebraska to the Pacific Coast, clays and claystone shales contain abundance of montmorillonite. Fig. 2.2 shows locations of expansive clay formations in the United States. Dashed lines are equivalent climatic ratings whereas numbers by points are reference numbers indicating reports of structural damage [8].

2.1.3 In the Kingdom of Saudi Arabia

One of the major geotechnical problems in Saudi Arabia, as has recently been noticed, is the swelling potential of the soils upon change in moisture content. In Saudi Arabia, the swelling soil formations vary in type and degree of swelling. To assess subsurface conditions, a general knowledge of topography, climate and geology is essential. Regions with expansive soil formations in Saudi Arabia are shown in Fig. 2.3. These areas have been investigated recently by a team of soil engineers from King Saud University, Riyadh K.S.A. [6]. The Al-Qatif area is being investigated in this study. Major structures founded in this area, have been seriously affected due to expansion of foundation soils [24].

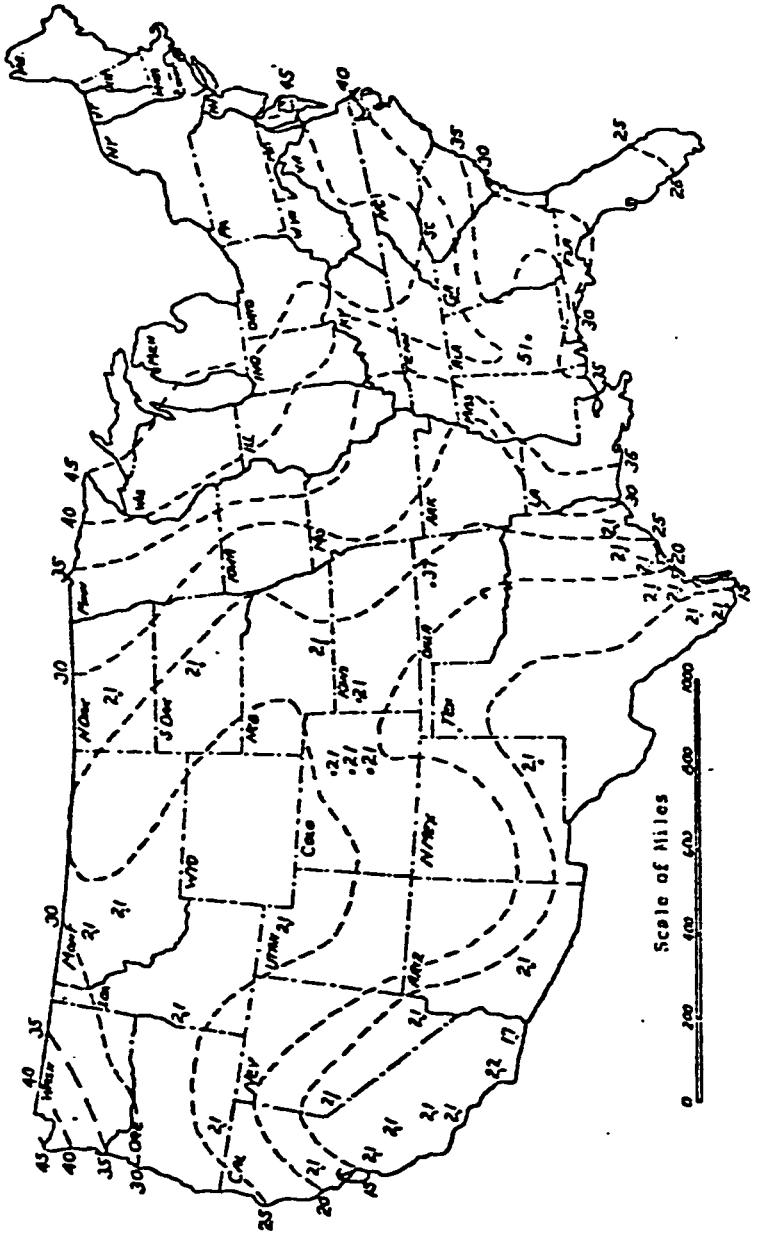


Fig. 2.2 : Locations of Expansive Clay Soil Problems in the United States (Gerald, 1974; Ref. 1)

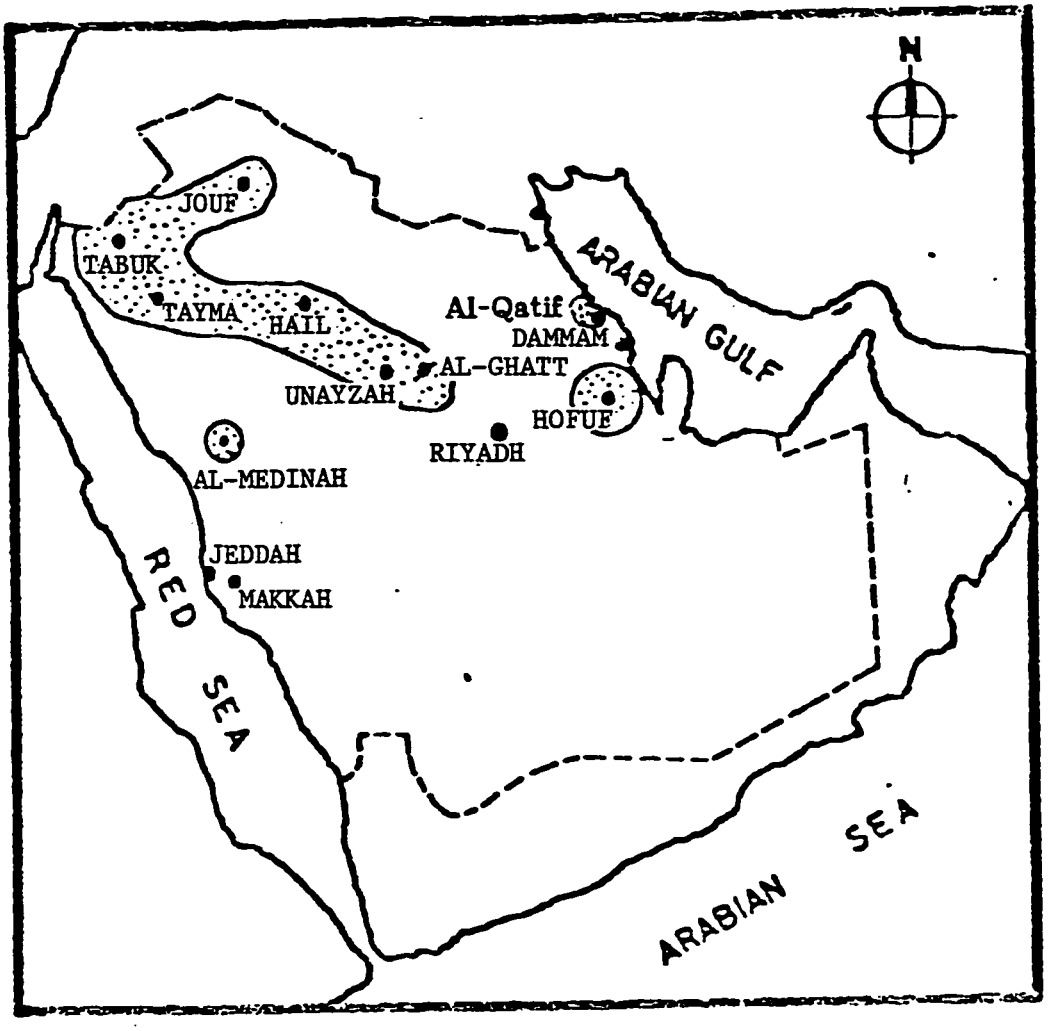


Fig. 2.3 : Tentative Distribution of Expansive Formations in Saudi Arabia (Dhowian, 1985; Ref. 3)

The subsurface soil conditions and stratification of the ground at regions with expansive soils are depicted in Figs. 2.4 & 2.5 which can be described as follows [5].

Medinah Region

Medinah city is located on a wide plain, approximately 10kms in diameter. The plain is formed by the accumulation of alluvial material in a basin to an elevation of approximately 600m above sea level. The area is covered by a thick layer of heterogeneous fill overlying a soft to medium stiff green-white clay formation identified as expansive in nature. This stratum occasionally contains calcareous rock fragments and sand layers. At greater depths a hard greenish clay is encountered.

Al-Ghatt Region

The top soil layer in this region (1 to 4m thick) is basically sandy silt with some gravels of various sizes and some gypsum intrusions changing gradually with depth to be clayey silt of moderate plasticity. Ghatt shales, which are part of the locally known Dhruma formations, has a very thin layered laminated structure. This formation possesses considerable amount of swelling potential. The in-situ natural moisture content of this shale is about 20% or more.

Tabuk Region

The subsurface profile in this region consists of three main soil

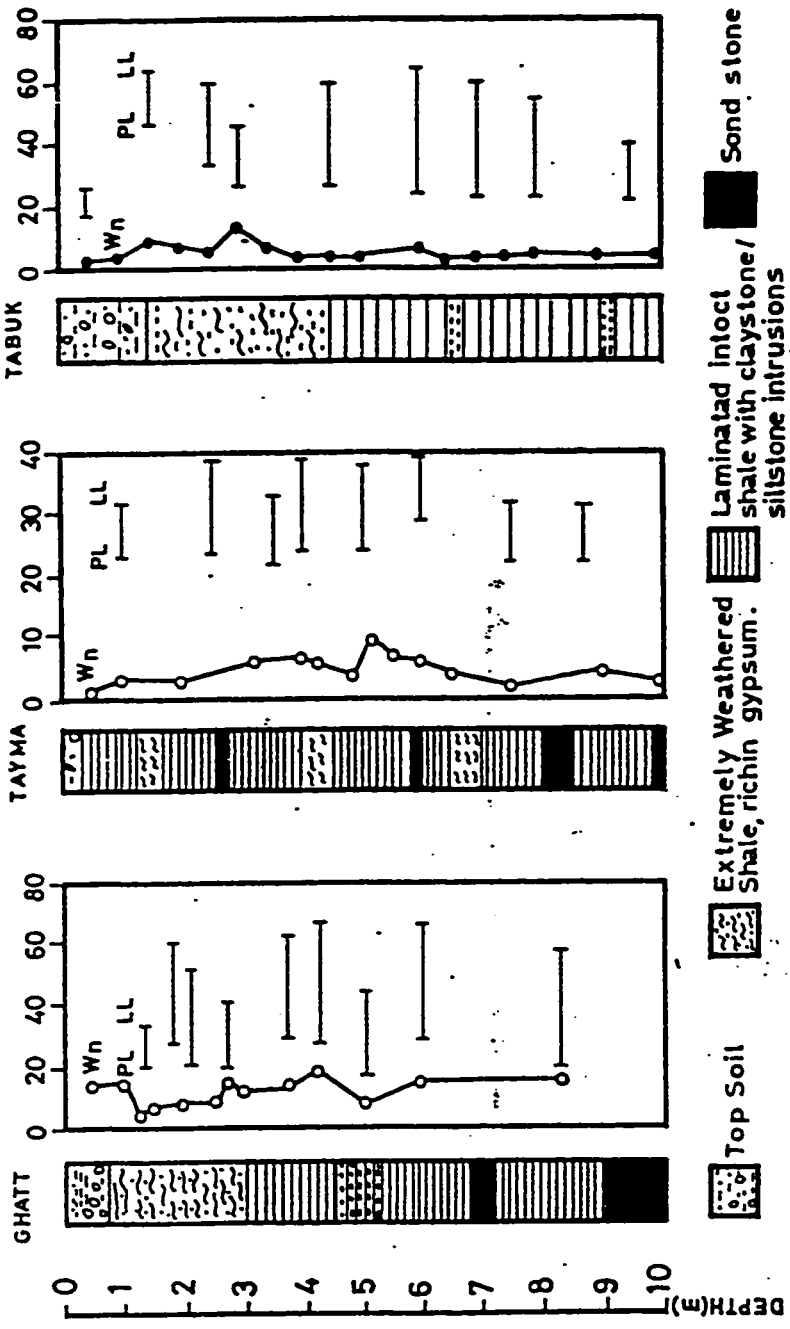


Fig. 2.4 : Typical Subsurface Profiles at Ghatt, Tayma and Tabuk

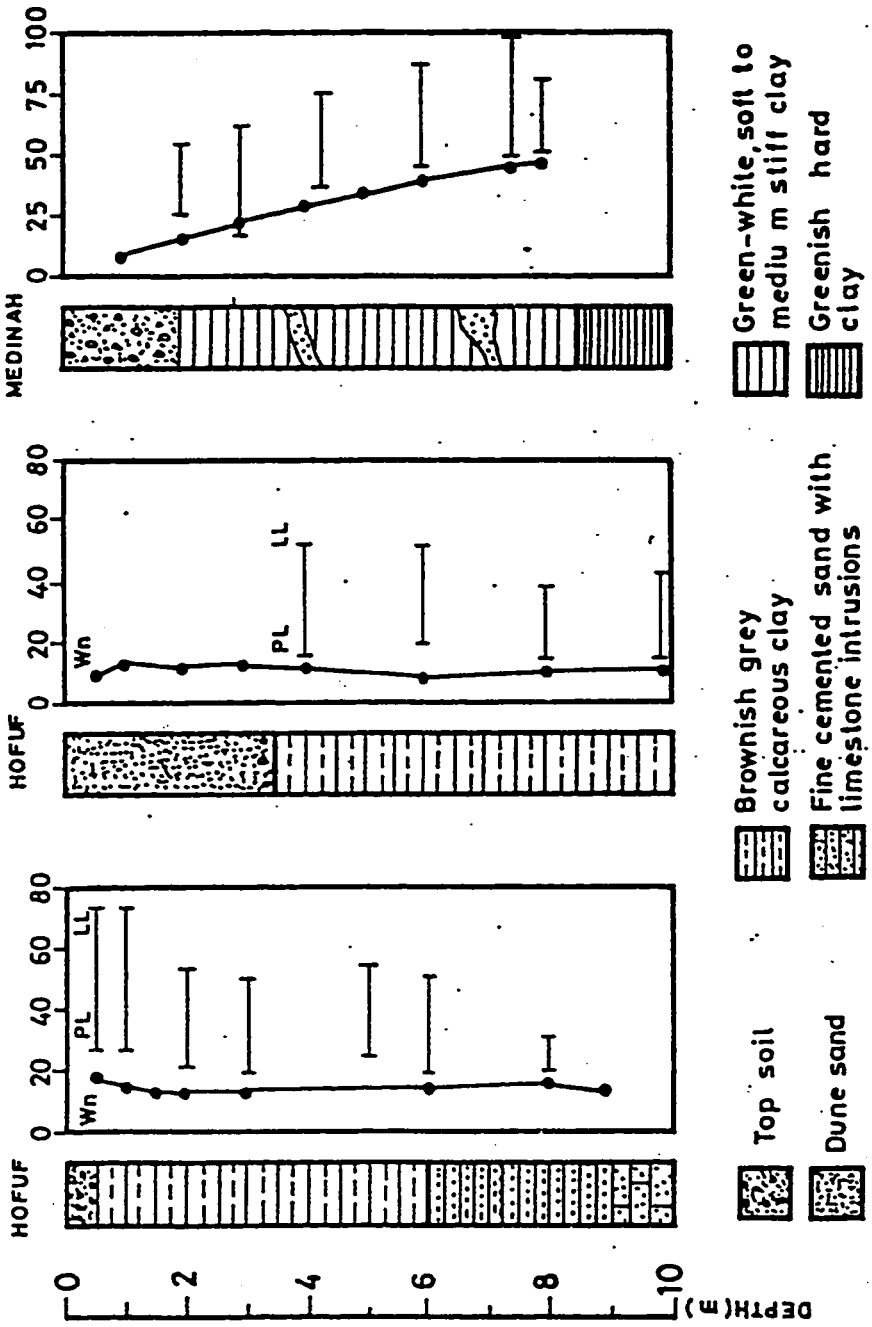


Fig. 2.5 : Typical Subsurface Profiles at Hofuf and Medina

layers: (i) surficial layer (1 to 6m thick) of fine to coarse medium dense gypsiferous silty sand with some gravels, (ii) brownish grey, hard fissured clayey silt with some gypsum (1 to 4m thick). This layer has high plasticity (LL=60 to 100%) and high swelling potential. (iii) this layer is part of the main shale formation. The soil of this layer is very hard greenish brown clayey silt (shale) interbeded with layers of sandstone and siltstone. The shale has medium plasticity (LL=40 to 60%) and moderate swelling potential. The underground water in Tabuk is at about 6m depth. Near farming areas, and at places where utility water is discharged, the underground water rises upto 3 to 4m below ground surface.

Tayma Region

Tayma is located at about 280 kms to the southeast of Tabuk. The subsurface conditions are somewhat similar to Tabuk region. The shales in Tayma are characterised by their high silt content and distinct laminations as compared to the shales in Tabuk region. The underground water in Tayma is at about 8m below the ground surface.

Hofuf Region

The soil profile in this region consists of top sand dune layer overlying clay/marl formation. The top layer varies in thickness from 1m to few meters and consists of uniform dune sand with some silt. The underlying layer (10-15m thick) consists of brownish grey

silty clay with some limestone nodules. It is called calcareous clay or marl. This layer is highly desiccated, impervious and salty having swelling potential. The underground water in this region is at about 8m below the ground surface.

2.2 Nature of Swelling Soils

2.2.1 General

Swelling soils are those soils which show a large volume change in response to soil moisture variations. Hard when dry, they sometimes are erroneously assumed to be excellent foundation soils; then after the structure is built, the clay slowly absorbs moisture and expands with a force sufficient to lift buildings and tear them apart. Although virtually all foundation soils rebound slightly upon unloading, the amount of such elastic rebound is minor compared to slower rebound that occurs as montmorillonitic clayey soils absorb free water [28].

Expansive soils can be classified into three groups, the first group comprises the sedimentary rock that contains montmorillonitic material, the second is sediment and the third is a product of weathering. Sedimentary rock or shale is originally laid down as a mud in a marine or lacustrine environment, and latter consolidated or squeezed into a dense rock from overburden loads. The main characteristics of expansive shales is that they are overconsolidated usually by erosion. They contain abundant clay minerals, and are

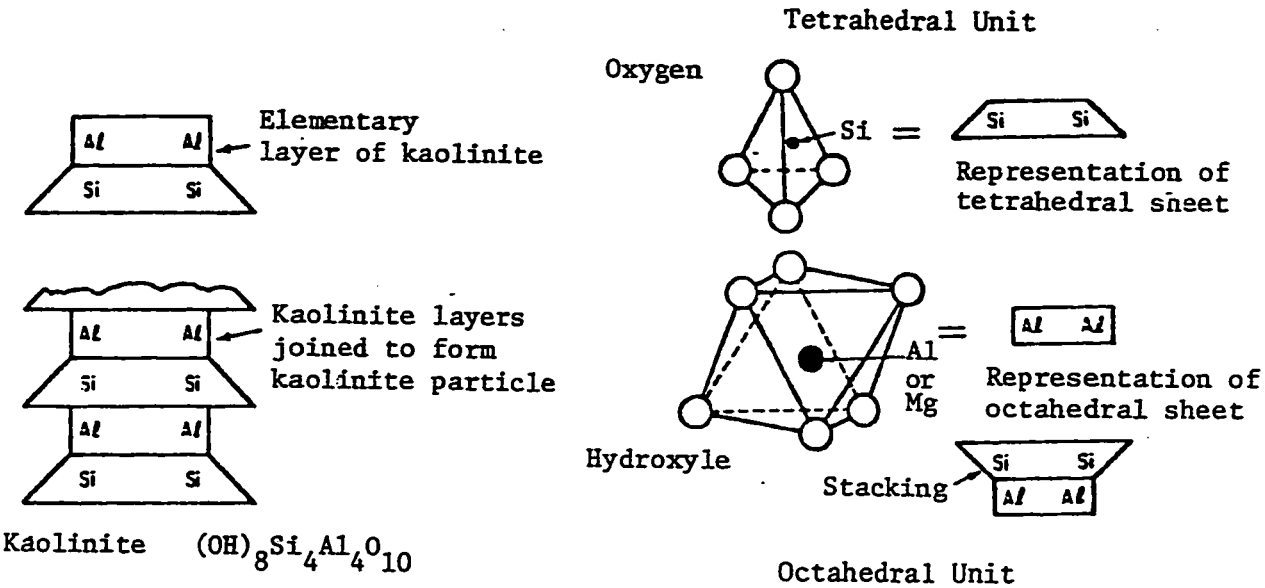
poorly cemented. Shales containing calcium carbonate are not expansive, and infact grade into lime stone.

The second type of expansive soils include alluvial and deltaic deposits or sediments that are derived from previous erosion of montmorillonitic clay shales. These geologically young deposits are overconsolidated by desiccation or drying.

The third source of expansive clay is formed by weathering and soil profile formation. In this type, sediments are formed and remain in-place, so it is local in occurrence. Areas with arid and semi-arid climates possess this category of expansive soils.

2.2.2 Clay Minerals

Clay minerals are essentially hydrous aluminium silicates or sometimes hydrous magnesium or iron silicates. The particles are very small and their behavior is dominated by electro-chemical activity characterised by a tendency to absorb certain cations and/or anions, and keeping them in an exchangeable state. Clay minerals are generally of crystalline form constituted from two fundamental building units, the tetrahedral unit and the octahedral unit as shown in Fig. 2.6. According to the arrangement of these building units in the crystal lattice, the clay minerals are divided into three main groups: Kaolinites, illites and montmorillonite. The structure of the crystal lattice also influences the magnitude of the negative charge and the extent of the affinity for water. The Kaolinite group has a



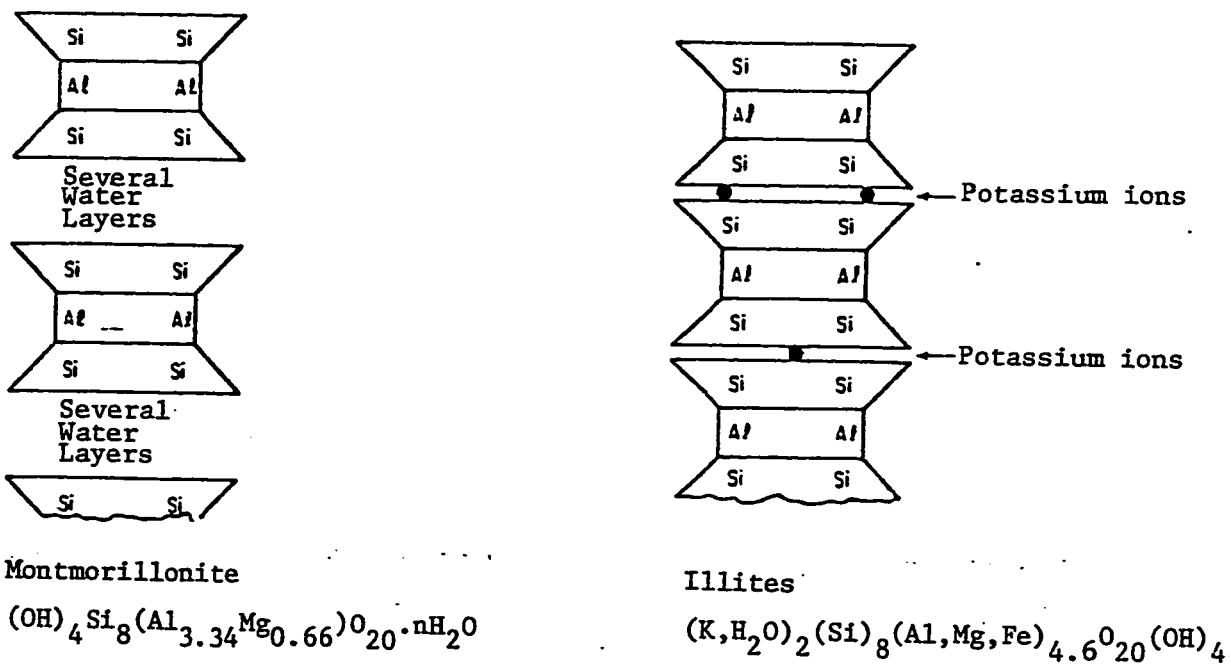


Fig. 2.6 : Structures of Important Clay Minerals (Young, Ref. 32)

structural block about 7 Å thick consisting of a sheet of tetrahedral units and a sheet of octahedral units. Hydrogen bonding between the blocks creates a relatively stable lattice structure not readily penetrated by water. Thus Kaolinites exhibit relatively low water absorption and have relatively low susceptibility to swelling under variations in water content. The illite group has a structural block about 10 Å thick, consisting of octahedral unit sandwiched between two oppositely oriented sheets of tetrahedral units. However, some of the silicon (Si^{4+}) portions in the tetrahedral units are filled with aluminium (Al^{3+}). This involves anions of lower valency replacing one of higher valency, and so potassium (K^{+}) ions attach themselves between the block in an attempt to compensate for the charge deficiency. The interlayer bonding of potassium ions is less stable than the hydrogen bonding, so illites exhibit a greater tendency for water absorption than Kaolinites and greater shrinkage and swelling characteristics. The montmorillonite group has a structural block similar to that of the illite but in addition to the substitution of aluminium for silicon in the tetrahedral units, some of the aluminium ions in the octahedral units are replaced by magnesium (Mg^{2+}). These changes result in a large negative charge which attracts water molecules and any available cations in the lattice. This interlayer water bond is very weak and unstable compared with the potassium ion bonding of illites. Thus montmorillonite exhibits very high water absorption and very high shrinkage and swelling characteristics [3].

Most clays are composed of more than one clay mineral, mixed in different amounts and various ways. Their expansivity is largely influenced by in-situ soil moisture content, the types of clay minerals in predominance, certain cation exchange capacities, the cations absorbed in the clay structures, and the composition of the ground-water supplying the exchange cations. The properties of the important clay minerals are summarized in Table 2.1 [7].

2.2.3 Swelling

Swelling is defined as the increase in volume of a clay soil as the water content increases. The amount of swelling depends upon the natural moisture content, type of clay minerals and their arrangement or orientation in the clay soil, as well as presence of organic matter. Everything being equal, swelling increases with increasing surface area of clay particles. Surface area of clay minerals depend more upon thickness of tabular particles than upon other dimensions. It is very high (50 - 840 sq.m/g) for thin particles of montmorillonite but very low (10 - 20 sq.m/g) for much thicker kaolinite particles. This is the one of the reasons that montmorillonite may swell the most out of all clay minerals [29].

Soils with random fabrics tend to swell more than soils with oriented fabrics. Disturbance or remolding of old natural clays may increase the amount of free swelling. Monovalent cations (e.g. sodium montmorillonite) will swell more than the divalent clays (e.g. calcium montmorillonite). Swelling like shrinkage is generally

Table 2.1 : Properties of Important Clay Minerals (Woodward-Clyde and Associates, 1967; Chen, Ref. 2)

Property	Kaolinite	Illite	Montmorillonite
Particle thickness	0.5-2.0 microns	0.003-01 microns	less than 9.5 Å
Particle diameter	0.5-4.0 microns	0.5-10 microns	0.05-10 microns
Specific surface (sq.m/g)	10-20	65-180	50-840
Cation exchange capacity CEC (meq/100 g)	3-15	10-40	70-80

confined to the upper portions of a soil deposit. Thus swelling damages light structures such as small buildings, floor slabs, highway pavements and canal linings [7].

2.2.4 Chemistry of Swelling

All clay minerals shrink and swell with changes in moisture content, these changes are minor compared to those of smectite group minerals (montmorillonite and saponite) which are able to adsorb water several times their weight. The diffuse double layer theory explains the influence of solution environment upon swelling in smectite. Calcium saturated smectite may swell from 10 Å to a maximum of 20 Å but Na and Li smectite theoretically swell to infinity. At high liquid limit, Na montmorillonite may have water films between 100 and 200 Å thick. Smectites in natural soils may not expand nearly as much as these values for several reasons [6]:

(i) they are generally interstratified with other nonexpansive clay minerals like micas, chlorites and chloritic interlayers etc.

(ii) they are seldom monoionic and are usually saturated with Ca, Mg and K rather than Na or Li.

The Diffuse Double Layer Theory

In a dry smectite clay, the negative charge is balanced by exchangeable cations surrounding the particles being held by electrostatic attraction. Cations in excess of those needed to neutralize the

electro-negativity of such clay particles and their associated anions are present as salt precipitates. When clay comes in contact with water, these cations and a small number of anions float around the clay particles. This is referred to as diffuse double layer. Because the adsorbed cations are responsible for a much higher concentration near the surfaces of clay particles, there is a tendency to diffuse away in order to equalize concentration throughout. The cation concentration decreases with distance from the surface of the particle. These cations are mainly responsible for expansion in montmorillonitic clays. The escaping tendency due to diffusion and the opposite electrostatic attraction lead to a cation distribution adjacent to a clay particle in suspension as shown in Fig. 2.7.

Water molecules are polar in character and some may be attracted directly to the particle surface, the positive pole being oriented towards the negatively charged surface. Others may be held directly on the particle surface by hydrogen bonding, or attached to hydrated cations which are themselves attracted to the negative clay surface. This water is more viscous than free water and is responsible for plasticity, swelling and shrinkage of clayey soils. Several theories have been proposed for the description of ion distribution adjacent to charged surfaces in clays. The Gouy-Chapman theory of the diffuse double layer as explained above has received the greatest attention [9].

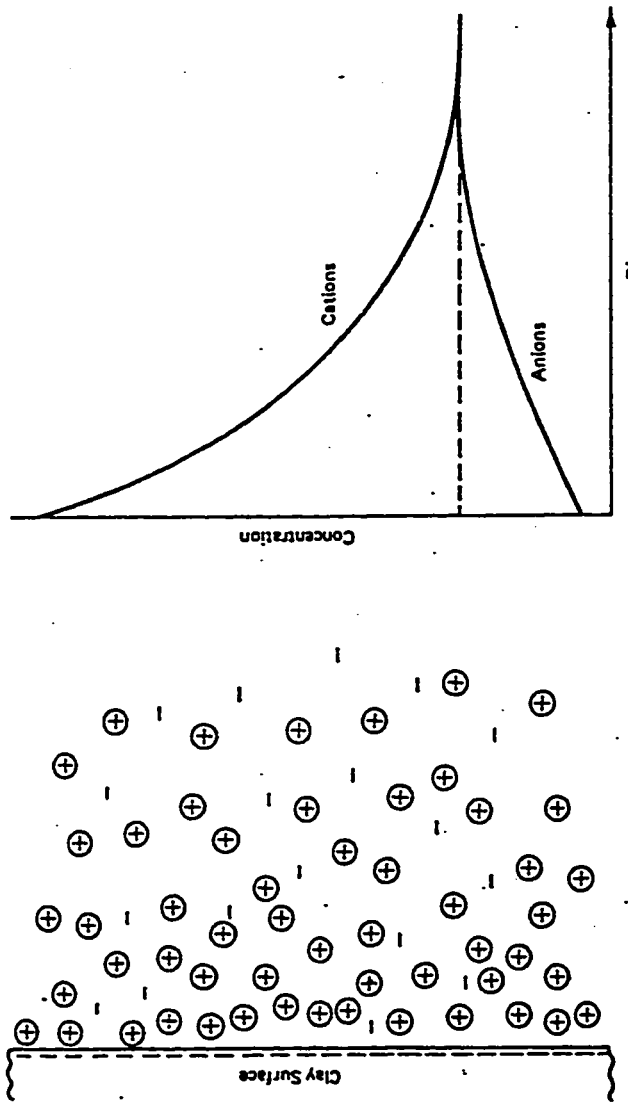


Fig. 2.7 : Distribution of Ions adjacent to an Expansive Clay Surface according to the Concept of the Diffuse Double Layer (Mitchell, 1976)

2.2.5 Shrinkage

The volume of a saturated soil is reduced as it is slowly dried. This phenomenon is known as shrinkage. Characteristics of shrinkage vary with the nature of soil. Total shrinkage increases with increasing water content. Shrinkage is also a function of percent clay in the soil, the kind of clay minerals, the mode of geological deposition and the degree of weathering. Sand and silt size particles reduce total shrinkage as they dilute the clay and decrease the volume of water held by the soil. High swelling clays containing the mineral montmorillonite possess high initial water content thereby increasing the amount of shrinkage. The cracks have an important role in water infiltration and movement. Water can be conducted rapidly into the cracks of an otherwise impermeable soil [28].

The effects of shrinkage of clayey soils may be of considerable significance from an engineering point of view. These cracks are the zones of weakness which can significantly reduce the overall strength of a soil mass and affect the stability of clay slopes and the bearing capacity of foundations. The desiccated cracks, which are usually found over deposits of soft clays, affect the stability of slopes and highway embankments constructed on these deposits [2].

Shrinkage and cracks are caused by evaporation from the surface in dry climates, and even desiccation of soil by trees during temporary dry spells in otherwise humid climates. One-dimensional vertical shrinkage might occur in a plastic soil at depth where the

overburden consolidates the clay, but the usual case is approximately three-dimensional shrinkage, accompanied by cracking. Where drying is not uniform, cracks will form in the wetter soil. A change in particle orientation occurs at the crack surface, and on redrying after wetting, the cracks will appear in the same places if the soil has not been otherwise disturbed. The number of cracks per unit area depends upon the clay minerals and the particle arrangement. A large number of cracks will be formed in flocculated clays, while in semioriented clay with higher cohesion, a few but relatively large cracks will result [28].

2.3 Recognition of Expansive Soils

Swell potential can be determined by the following parameters.

2.3.1 Atterberg Limits

Expansive soils can be identified by evaluating their index properties (Holtz and Gibbs, 1957). The plasticity index is the most common indicator of potential expansion. Simple index tests are used to determine the Atterberg limits. An estimate of the degree of expansion can be related to these limits. As an example, linear shrinkage, liquid limit, and plasticity index greater than 8%, 30%, and 12% respectively were among the first criterion proposed for the recognition of an expansive soil (Kanty and Brink, 1952). Table 2.2 & Table 2.3 contain an improved guide to the determination of potential expansion for various values of Atterberg limits (Gerald, 1974).

Table 2.2 : Classification of Swelling Soils based on Index Tests (Gerald, 1974; Ref. 1)

Percentage Passing #200	Liquid Limit (%)	Plasticity Index (%)	Shrinkage Limit (%)	Degree of Expansion
<30	<30	<18	>15	Low
30-60	30-40	15-28	10-16	Medium
60-95	40-60	25-41	7-12	High
>95	>60	>35	<11	Very High

Table 2.3 : Proposed Expansivity Rating System for Expansive Soils in New South Wales (Tadanier, 1984)

Linear Shrinkage	Shrinkage Index	Plasticity Index	Liquid Limit	Shrinkage Limit	Expansivity Rating
0-8%	<25%	<18%	<35%	>14%	Low
8-13	25-35	18-25	35-45	12-14	Medium
13-18	35-50	25-35	45-60	10-12	High
>18	>50	>35	>60	<10	Very High

2.3.2 Percentage and Type of Clay Minerals

The mineralogical composition has an important bearing on the swelling potential of expansive soils. The amount and the type of clay minerals are very important in the study of these type of soils. Hence, it is claimed by clay mineralogists that the swelling potential of any clay can be evaluated by the identification of its constituent minerals. Presently, X-ray diffraction method and Scanning Electron Microscopy (SEM) are widely used for the quantitative and qualitative estimation of clay minerals in a soil sample respectively.

X-ray diffraction method used in determining the percentage of various minerals in a colloidal clay consists essentially of comparing the ratios of the intensities of the diffraction lines from different minerals with the intensities of lines from the standard substances prepared by the Joint Committee for Powder Diffraction Analysis (JCPDA) [1].

Scanning electron microscopic examination of clay minerals offers a direct observation of the material. The main purpose of the microscopic study is to determine mineralogic composition, texture and soil fabric. Ravina in 1973, extensively studied the mineralogical composition of expansive clays by the use of SEM [9]. He showed that the nonswelling clays appear as flat, relatively thick plates, while montmorillonitic clays have a crinkly ridged honeycomb-like texture. It might be possible to evaluate some properties of expansive soils by observing the degree of crinkling and interparticle bonding [10].

2.3.3 Swelling Tests

Once a potentially expansive soil has been identified and a qualitative indication of potential swell has been made, quantitative characterization is necessary to estimate the amount of anticipated volume change. Reliable estimates of heave are prerequisite for the selection of treatment alternatives to minimize the volume change or preparation of foundation design to accommodate the volume change. Techniques available for quantitative characterization of expansive soils fall into three categories: namely, free swell test, odeometer tests (swelling pressure and percentage swell tests) and empirical relationships.

Free Swell Tests

This is a simple identification test developed by U.S. Bureau of Reclamation. This test is conducted by placing a known volume (10 cm^3) of dry soil in a 100 cm^3 graduated jar filled with water and noting the equilibrium swelled volume. Free swell is defined as:

$$\text{Free Swell} = \frac{(\text{Final Volume}) - (\text{Initial Volume})}{(\text{Initial Volume})} \times 100\ \%$$

The free swell test is very crude and was used in the early days when refined testing methods were not available. Experiments conducted by (Holtz and Gibbs, 1956) indicated that a good grade of high swelling commercial bentonite (mostly Na-montmorillonite) will have free swell values as high as 2000%. Holtz suggested that soils

having free swell values as low as 100% may cause considerable damage to light structures. Soils with free swell values less than 50% have been found to exhibit small volume changes [13].

Swell Percentage or Unrestrained Swell Test

In 1962, Seed defined swell potential or swell percentage as the percentage of swell of a laterally confined sample on soaking under 1-psi surcharge. This test essentially involves the direct measurement of the vertical rise of a soil sample placed under 1-psi surcharge and in contact with water inside a consolidometer. After the specimen has reached its maximum volume change, it is loaded vertically by increasing the loads in small increments on the lever arm. The swell percentage defined as the vertical rise of soil specimen to its original height, measured when no further swelling takes place under 1-psi load [2]:

$$S_{\%} = \frac{\Delta H}{H} \times 100\ \%$$

Where,

$S_{\%}$ = Swell percentage

ΔH = Change in height due to saturation

H = Original height of the specimen

Swelling Pressure Test or Restrained Swell Test

The swelling pressure test can be conducted by confining a soil specimen in a consolidation ring and applying a pressure equal to the effective overburden pressure. Water is then added to the specimen. Total load required to bring the sample back into its original volume divided by the cross-sectional area of sample is defined as swelling pressure.

Swelling pressure values of 20-30 KN/m^2 are considered to be low, and a value of 1500-2000 KN/m^2 is supposed to be very high. Swelling pressure generated by smectite group minerals may approach as high as 1000 KN/m^2. Table 2.4 shows the classification of swelling soils based on different types of swell tests [31].

Soil Suction Test

In theoretical analysis, the total suction can be considered to consist of the osmotic (or solute) potential, gravitational potential, and the matrix or capillary potential. In engineering practice, however, it is considered satisfactory to conduct laboratory analysis by simulating the actual capillary potential in the soil. The capillary potential can be considered as being equivalent to the negative pore pressure in a soil mass. The capillary potential of an unsaturated soil is often identified in terms of its soil suction.

Soil suction is expressed in a term designated as pF which is log of the equivalent capillary rise in centimeters of water. Thus a

Table 2.4 : Classification of Swelling Soils based on Swell Tests (Gerald, 1974; Leopoldo, 1984)

Swell Potential as Percent	Total Expansion (1-psi load) (%)	Swelling Pressure			Degree of Expansion
		U.S. Customary (Tsf)	S.I. Units (KPa)	Metric Units (Kg/cm^2)	
0-1.5	0-10	<2.05	<196	<2	Low
1.5-5.0	10-20	2.05-4.1	196-392	2.0-4.0	Medium
5-25	20-35	4.10-7.2	392-687	4.0-7.0	High
>25	>35	>7.2	>687	>7.0	Very High

pF of 2 represents 100 cms of hydrostatic heads (205 psf) and so on. The amount of soil suction of a soil sample at equilibrium with water is zero. Upon drying, the soil suction rises rapidly. At oven drying condition, the value may be several thousand atmospheres.

The use of thermocouple psychrometers is one of the simplest and most reliable procedures available. The psychrometer measures the relative humidity of a soil specimen in a sealed container. Soil suction is directly related to humidity; when calibrated for various relative humidities (or soil suctions), the psychrometer output can easily be converted to soil suction in any convenient units [28].

Empirical Relationships

Because of the problems encountered in the performance of structures founded on expansive soils, several attempts have been made to develop reliable methods for their identification. These methods are based on a correlation between soil expansion and some factor that is directly related to clay mineral characteristics such as plasticity index, shrinkage limit, percentage less than 2 μm and activity (Holtz and Gibbs, 1956).

Swelling tests on artificial sand-clay mixtures were used as the basis for the development of one method (Seed, Woodward, and Lundren, 1962). The standard for comparison was the expansion of laterally confined specimens, prepared at optimum water content using standard AASHTO compactive effort, and allowed to swell under a

surcharge of 1 psi. An excellent correlation was found between the expansion and compositional factors that expressed both the type and amount of clay, according to [8]

$$S = 3.6 \times 10^{-5} A^{2.44} C^{3.44}$$

Where S is the percent swell of a sample prepared and tested under the specified conditions, A is the activity and C is the percent clay size (< 2 μm).

Further empirical studies indicated that for natural soils the swelling potential could be related to the plasticity index with an accuracy of ±35%, (Seed, Woodward and Lundgren, 1962) as follows;

$$S = 2.16 \times 10^{-3} (PI)^{2.44}$$

A slight different relationship (Ranganathan and Satyanarayan, 1965) has been found to better classify the swell potential of some soils, especially the black cotton soils in India. The swell activity (SA) was used to express compositional factors relating to swelling and is defined by:

$$SA = \frac{SI}{A}$$

Where SI is the shrinkage index, given by the liquid limit minus the shrinkage limit. The resulting relationship is;

$$S = 41.13 \times 10^{-5} (SI)^{2.67}$$

Similar relationships have also been proposed by Nayak and Christensen (1971) as follows:

$$S = B(SI)^P$$

in which B = 1/1.63 and p = 1.17.

2.4 Factors Affecting Swelling

The swelling potential of clay soil depends on the following parameters [7].

1. Mineral Type and Amount - Qualitative & quantitative estimation of different types of minerals present in a soil sample, can be done by optical and electron microscopy, x-ray diffraction, differential thermal analysis, infrared analysis, dye adsorption and chemical analysis techniques. The swell also depends upon the exchangeable ion's capacity. A smaller particle size allows more water adsorption per unit volume of clay particle.

2. Soil Structure - The stress history of a soil specimen also influences swell. Undisturbed or cemented expansive soils have high resistance to deformation thereby imparting large swelling pressures. Remolding undisturbed clay reduces initial hardness at high shear strength, and preferentially alignes the flat grains normal to the compacting forces. Soils with lined particles, swell more in directions normal to platelets than in directions parallel to platelets. Static compaction allows more swell than impact or

dynamic compaction.

3. Initial Moisture Content - In testing undisturbed samples, care should be taken in selecting the sample with the most critical moisture content. Usually, tests should be performed on the driest sample since dry clays swell more than wetter ones because of the direct relation between water content and suction pressures.

4. Initial Dry Density - Density is an important factor affecting swelling characteristics of soils. Dense clays will swell more when they become wetted, compared with the same clay at a lower density and same initial water content.

5. Surcharge Pressure - Surcharge loads reduce the swell to such a degree that expansion can be held to zero in the presence of free water. Allowance of a small amount of swell greatly reduces the swelling pressure.

   A small surcharge load in the range of 0.35 to 1 psi has been suggested for a seating load in the swell test. Since swell is very sensitive to changes in the lower ranges in pressure (less than about 1 psi), the use of low surcharge pressure may lead to erratic and erroneous results. Since most footing foundations can exert a pressure of about 1,000 psf (0.5 kg/cm^2) on the soil, it is recommended that this value be used for a surcharge load.

6. Time Allowed for Swell - The time required for the soil to reach its maximum swell potential may vary depending upon the initial

moisture content, density, permeability and thickness of the sample. For remolded samples, generally 24 hours is sufficient to obtain 95% of the total available swell. At the same time, for undisturbed high density clay shale, it may require several weeks to achieve complete saturation. For remolded samples, the initial added water must be evenly distributed. This requires a minimum curing time of 6 hours for reproducible results.

7. Pore Fluid - A high pH encourages cation disassociation. High salt concentrations in the free water may cause less water sorption and less swelling.

2.5 Methods of Treatment

Based on the economics and practicality of the operation, there are a number of methods which can be used to minimize heave in expansive soils. These methods may include compaction, ponding, moisture barriers, replacement and chemical stabilization. Any of these techniques may be applicable depending on the soil conditions, material, type of construction and accessibility of equipment at the construction site. The success of any stabilization method depends upon the consideration of all conditions and an understanding of the application and limits of that particular method [19].

Compaction

One of the most practical and economical methods of controlling heave of swelling soils is compaction. Compaction is used in

conjuction with replacement or alone to control expansion and shrinkage of near-surface soils. In general, compactive effort should be limited in order to limit the suction but should be high enough to maintain adequate bearing pressures that develop in compacted soils. Compacted soils can be placed above moderately expansive clays to serve as a surcharge which will reduce expansion in near-surface soils. Floor slabs, and in some cases, lightly loaded interior footings can then be placed in the fill. Gromoko in 1974 recommended compaction at 2-5% above optimum moisture content [8].

Ponding

Covering a construction site with water prior to construction is known as ponding or prewetting. The purpose of ponding is to allow desiccated foundation soils to swell prior to placement of structure. This method has been successful in a few cases. However, wetting the foundation subsoil by ponding may require many months or even years to increase the water content to the required depths unless the clay contains a fissure system to aid water percolation through the soil. After prewetting, 4% or more of hydrated lime is often mixed with the top layer of clay to reduce its plasticity and make it more workable. As an alternative, the application of a 4-6 inch thick layer of coarse gravel, sand or granular soil on top of the area will aid considerably in providing a good working surface during and after prewetting. This layer has advantages in reducing evaporation, providing a minor surcharge, as well as making a level uniform

subgrade.

Moisture Barriers

Moisture barriers may be constructed around the perimeters of slabs-on-grade to minimize moisture variations beneath the slab's perimeter. Recent studies suggest that vertical trenches about 15 cm wide and 1.5 m deep filled with gravel, lean concrete, or mixtures of granulated rubber, lime and fly ash reduce the long term effects of differential movement around slab's perimeter. In one field study on residential slabs (O'Neill, 1977), perimeter movements (heave) in a CH-soil were 5-8 cm when the surrounding subgrade was flooded, but movements stabilized in 1-2 years. Movements on similar foundation without barriers were somewhat higher but were far variable around the perimeter.

Replacement

Expansive clays can be replaced by less expansive materials when such clays are close to the surface. low expansion clays are preferable to granular soils in this respect because granular soils are conducive to collecting water on the surface of expansive clays that were not excavated, either from the air through hydrogenesis or through surface seeps.

Chemical Stabilization

Chemical stabilization of expansive soils by chemical stabilizers

such as lime, potassium and cement etc., has been tried for many years with varying degrees of success. Organic chemicals like fly ash and other compounds are also useful to reduce heave in expansive type soils [30]. Among chemical stabilization, options are:

1) Intermixing or blending small lime additions (from 2-8%) with the expansive clays. The primary effect of this approach is to decrease significantly the liquid limit, plasticity index, maximum dry density and swell, and to increase the optimum water content and strength of such soils. The mixing of lime and subsequent compaction should begin immediately after adding water to the soil (Croft, 1967). Woodward and others in 1965 found that attempts to distribute lime with water in drill holes through natural soils was not very successful. It was necessary to excavate the soil, mix it directly with lime [8].

2) Addition of potassium or other active cation to expansive clays, making them non-expansive. This is typically done by pressure injection of aqueous solution. Chemical injection is particularly adaptable to homogeneous clays, clay stones and clay shales. This method is increasingly difficult to perform as soil's variability increases. The process is particularly useful as part of the remedial work on existing heave-damaged structures. It may be used to treat soils beneath and around piers, piles and caissions, where it may reduce uplift and drag forces imparted to the foundation through skin friction [15].

3) Lime slurry pressure injection. This differs from the two processes described above in that a lime suspension (rather than solution) in water is injected under pressure. The lime is deposited on surfaces of natural soil blocks, in time may penetrate upto 2 in. into the blocks, and inhibits moisture mitigation into and out of them. The technique provides good stabilizing results in many cases, but sometime has only limited long-term effects. Lime slurry pressure injection is reasonable for use in any area where it has a demonstrably effective track record. In such cases it is the most economical chemical stabilization technique [30].

2.6 Foundations on Expansive Soils

Prior to arriving at a final foundation design on an expansive clay site, at least four distinct steps must be taken by the designer [19].

a) Identification- Determine whether potentially expansive clays exist.
b) Classification- If potentially expansive clays do exist, what degree of attention needs to be paid to them
c) Quantification- If clays are sufficiently prone to volume change, a rational assessment of numerical values of swell potential or probable vertical movement should be made.
d) Evaluation of Design Alternatives- Compare alternative foundation designs to determine the most suitable and economical

design compatible with the predicted vertical movement.

Three basic approaches can be followed:

1) Alter the condition of the expansive clay.
2) Bypass the expansive clay by isolating the foundation from its effect, and;
3) Provide a shallow or deep foundation capable of withstanding differential movements and mitigating their effect in the super-structure.

CHAPTER 3

GEOTECHNICAL INVESTIGATION

3.1 Sampling and Preservation

Sampling is the act of extracting relatively small pieces (samples) of soil or rock from the mass of soil or rock. To perform this act several more or less sophisticated techniques have been developed. Classifying a sample as "undisturbed" generally is assumed to mean that the sample, when tested in the laboratory will give data that can be used with reasonable accuracy to predict the field action of the soil mass. With proper techniques and effort it is often possible to obtain undisturbed samples of cohesive soils [16].

A site located in the area of Al-Qatif near the King Fahd Hospital at the Anak exit of Dhahran-Jubail highway was chosen to take undisturbed soil samples. Structural damages in the area due to expansive soil were reported.

The subsoil conditions obtained from previously drilled boreholes are shown in Fig. 3.1 to 3.3. These soil profiles have been drawn based on borehole logs prepared by Test Consultant for Al-Qatif Housing Project. It is clear from these subsurface profiles that the area is covered by thin layer of heterogeneous fill overlying a greenish brown clay formation.

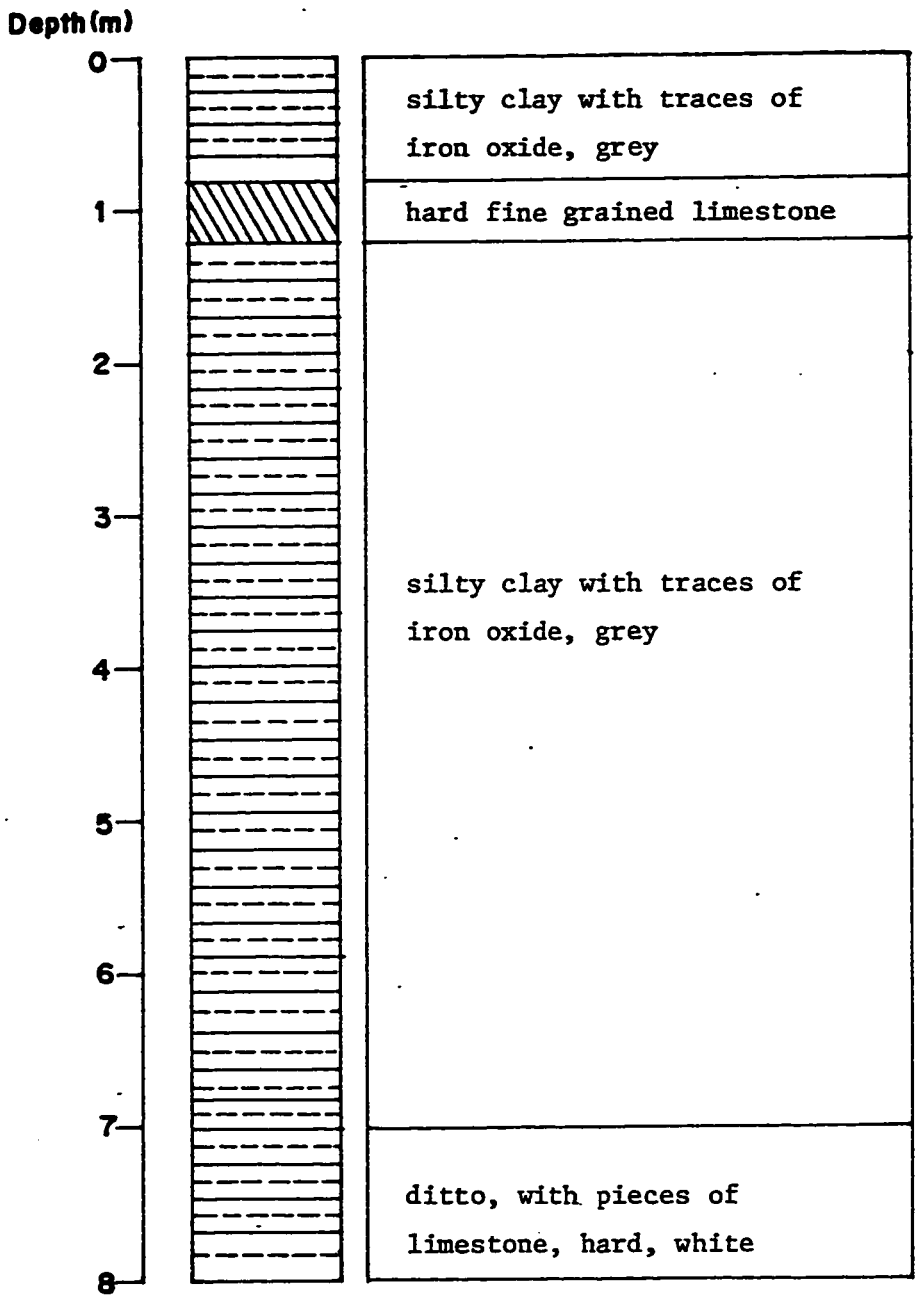


Fig. 3.1 : Subsurface Soil Profile at Boring # 01

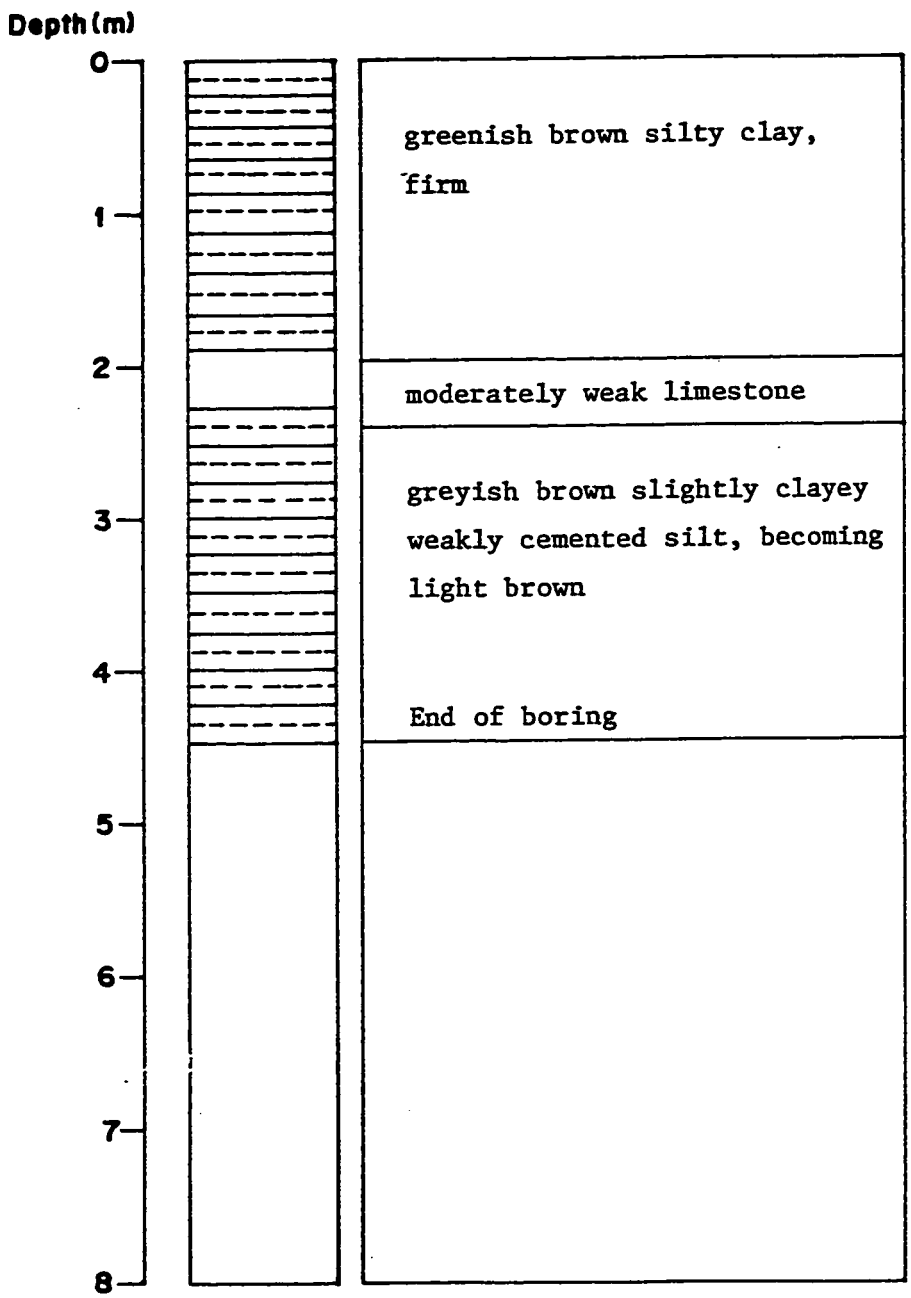


Fig. 3.2 : Subsurface Soil Profile at Boring # 02

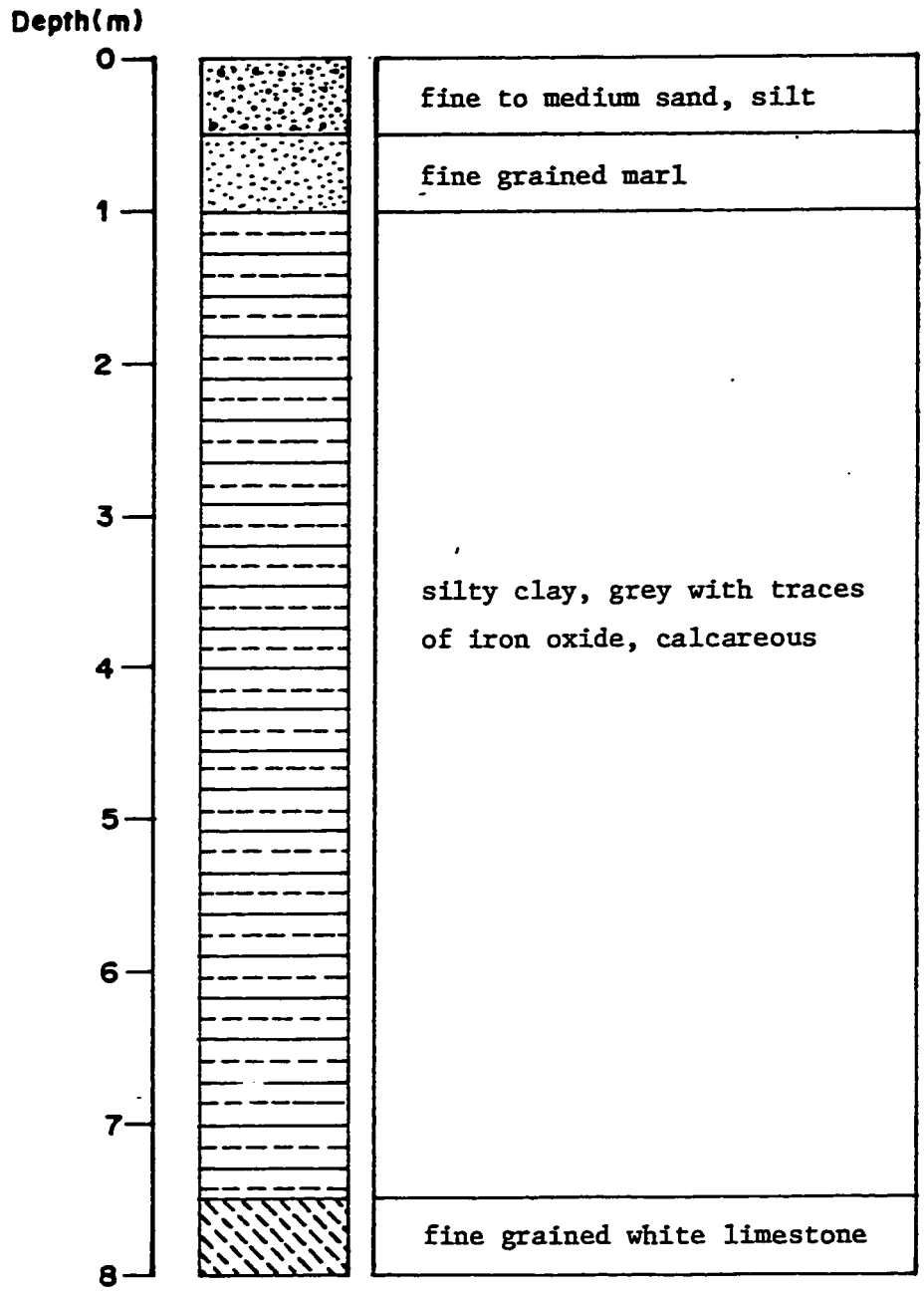


Fig. 3.3 : Subsurface Soil Profile at Boring # 03

The soil profile shown in Fig. 3.1 indicates that the ground mainly consists of greyish silty clay with traces of iron oxide. This formation is classified as calcareous clays. The calcareous clays are underlain by a thin layer of hard fine-grained limestone. From 1.5 m to 7.0 m a thick greenish-brown silty clay with traces of iron oxide was found.

Another soil profile at test site is shown in Fig. 3.2. The subsurface profile indicates that the area at this location is covered by a 2.0 m thick layer of firm greenish brown silty clay underlain by a moderately weak limestone. This stratum below thin layer of limestone contains greyish brown silty clay. Details below 4.5 m depth were not available as the boring was not continued to greater depth.

At the study site at one location, the area is underlain by a 6.5 m thick layer of silty clay with traces of iron oxide. The upper part of this formation basically consists of fine to medium sand, silt and fine-grained marl. Typical subsurface profile at this location is shown in Fig. 3.3. The uppermost zone of this part is extremely weathered and transformed to a gypsiferous clayey silt. A weak bedrock of fine-grained white limestone was found below 7.5 m depth.

According to Test Consultants for Al-Qatif Housing Project, the underground water table in Al-Qatif area at the study site was 6-8 m from the ground surface. The high natural water content of surface soils was attributed due to capillary rise.

Undisturbed block samples were taken from excavated test pits which were dug to the level of the stratum to be sampled. Rectangular test pits upto 3 m deep were excavated. One of such test pits is shown in Fig. 3.4. The sides of the blocks to be removed were marked and excavated carefully to within 15 mm (1/2 inch.) or so of the area marked. Different sizes of blocks ranging from 20 cm to 40 cm high were excavated from the bottom & sides of the trenches. Fig. 3.5 shows how a block sample was taken from the bottom of a test pit. Samples were trimmed and coated with paraffin-wax at the site. The blocks were then loaded in a vehicle and transported to the soil lab. at KFUPM. The blocks were sealed with tape & stored in a laboratory humid room to inhibit evaporation of natural moisture.

Three test pits from four different locations were used for taking block samples as shown in Fig. 3.6. The blocks from a particular location were designated a particular block number. The depth of the test pits is shown in meters but the horizontal distances which were not in one line, are not drawn to the scale. Block # 01 and # 02 were taken from test pit # 01 which was open to air, while block # 03 and # 04 were collected from test pit # 02 and # 03 respectively.

3.2 Classification Tests

3.2.1 Grain Size Analysis

ASTM D-1140 procedure was followed to determine the amount of material in soil samples finer than #200 sieve. The steps involved

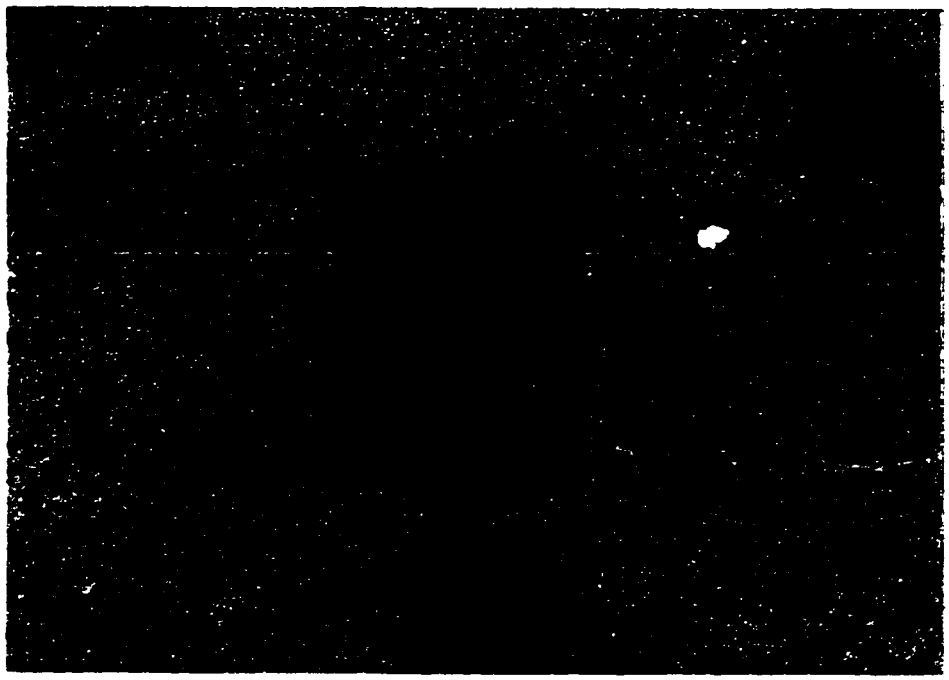


Fig. 3.4 : Test Pit Excavated for Block Sampling

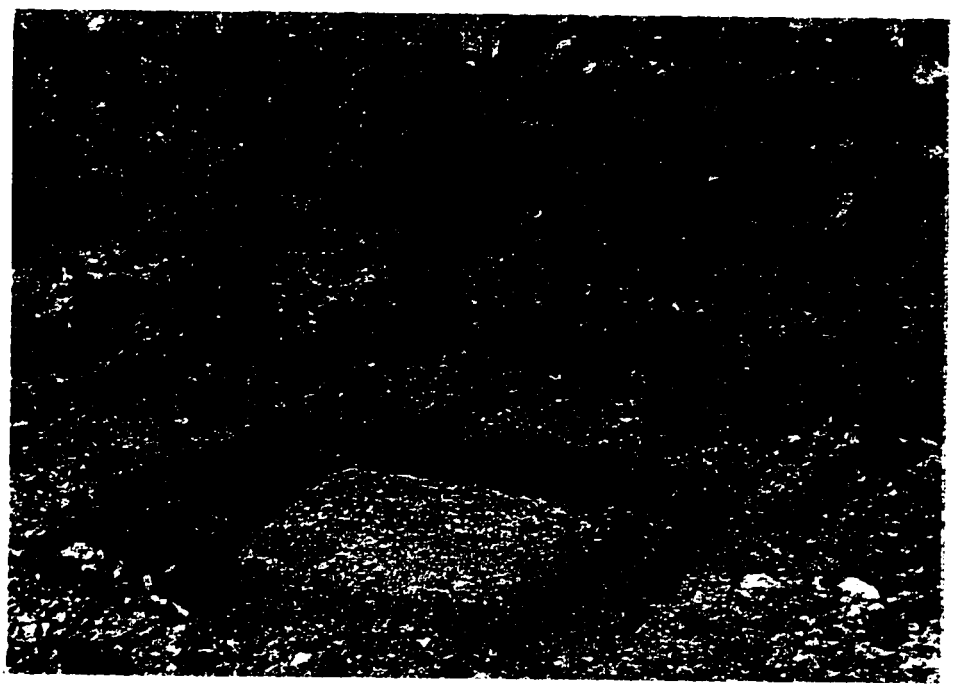
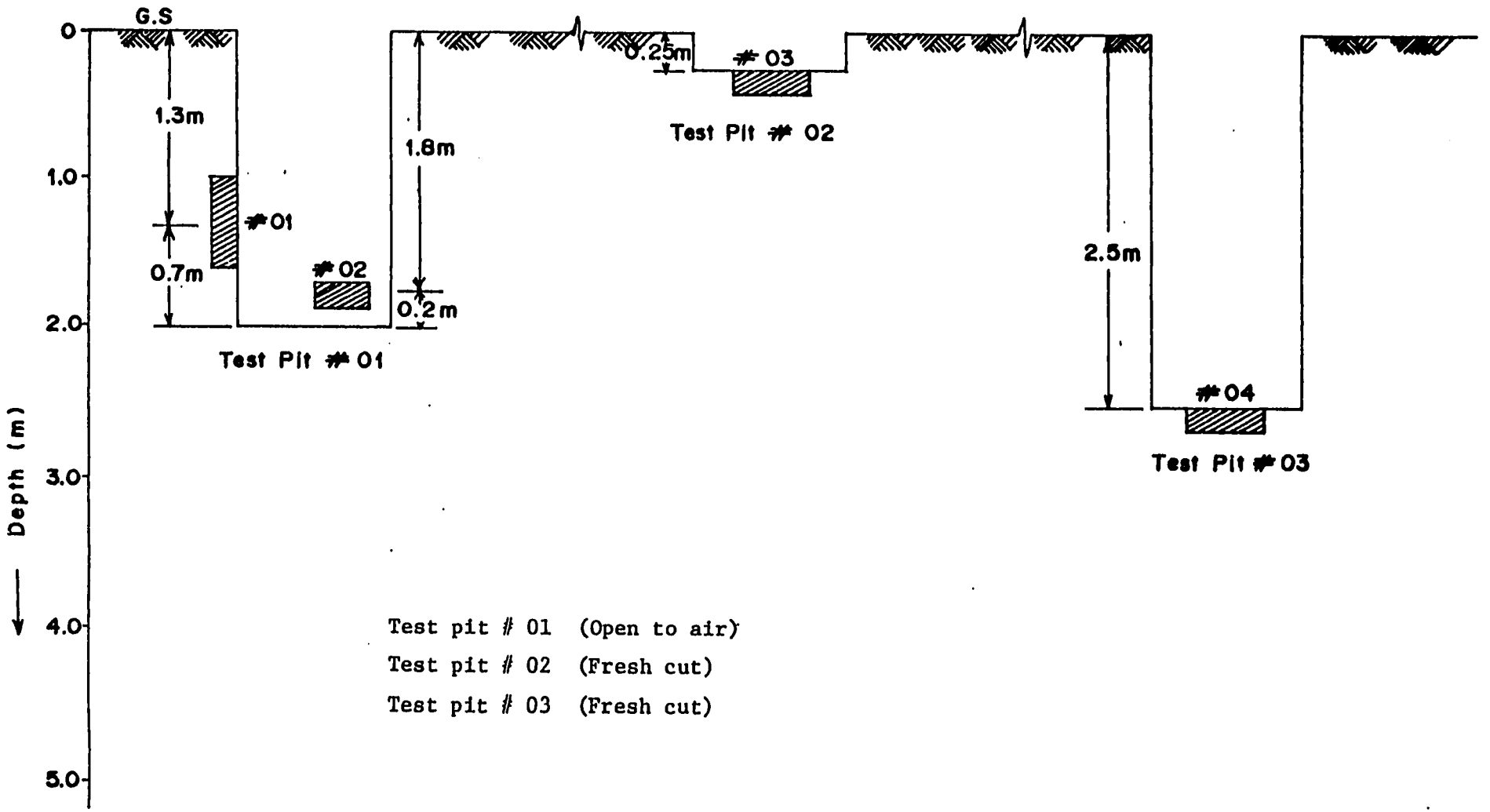


Fig. 3.5 : Technique Used for Block Sampling



Test pit # 01 (Open to air)

Test pit # 02 (Fresh cut)

Test pit # 03 (Fresh cut)

Fig. 3.6 : Description of Undisturbed Expansive Soil Samples taken from Different Locations in Al-Qatif area (K.S.A.)

in this method are as follows:

1) The test sample is dried to a constant weight at a temperature not exceeding 110 ± 5°C. The oven-dry weight of the sample is calculated from the moist weight and the moisture content.
2) The dried sample is placed in a container, sufficient clean water is added to cover it, and allowed to soak for 12 hours.
3) The contents of the container are agitated vigorously and water is poured immediately over the nested sieves, arranged with the coarser sieve at the top. The process was repeated adding clear water to the container to cover the sample, until the wash water is clear.
4) The washed material retained on the nested sieves is dried in a container to a constant weight at a temperature not exceeding 110 ± 5°C. The dry material retained on nested sieves is weighed.

The material passing #200 was analyzed by the hydrometer method following ASTM D-422 procedure. For most of the soil samples analyzed in this study, the material was passing #200. Fig. 3.7 shows the grain size distributions of collected samples which lie within the range of silty-clay soil. The sieve washing analysis was preferred since the natural water content of the soil mass was upto 58%. Later this data was combined with the hydrometer analysis to get complete curves. The grain size distribution curves in Fig. 3.7 indicate comparatively a narrow range. Clay fraction (< 2 μm) is

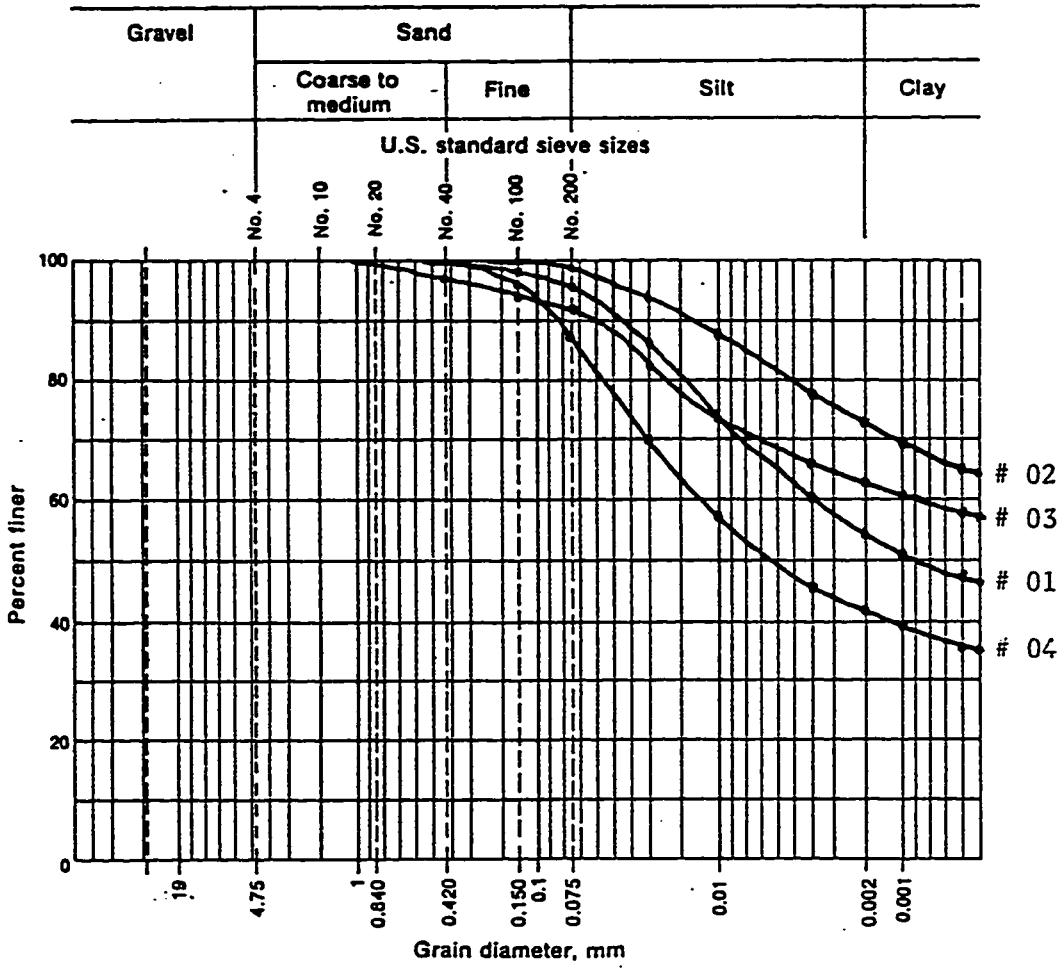


Fig. 3.7 : Grain Size Distribution of Al-Qatif Soils for samples from four blocks based on Unified Soil Classification System

85-98%. Sand was negligible, silt and clay particles were in predominance. No gravels were found in any of the blocks taken from the site.

3.2.2 Unit Weight and Natural Water Content

Unit weight and natural water content were determined according to ASTM D2937 and ASTM D2216 respectively. The two important properties for different samples from the four blocks are listed in Table 3.1.

The natural water contents of the undisturbed soil blocks were very high and varied from 28.4%-59.7%, resulting in a high bulk density and comparatively low dry density as summarized in Table 3.1. Although block #01 & #02 were sampled from the same test pit as shown in Fig. 3.6 but there is a remarkable difference in their water contents. Samples from block #01 are the driest since the test pit # 01 was open to air upto that level while samples from block # 02 possess higher water contents as this block was taken after some excavation at the bottom of test pit # 01. Soil samples from block # 04 possess the highest water contents than all others because this undisturbed block was taken from a fresh cut and the sampling was done at a greater depth.

3.2.3 Determination of Atterberg Limits

The liquid & plastic limits were determined following the ASTM D-4318 procedures, while determination of shrinkage limits was

Table 3.1 : Some Physical Properties of Soil Samples Studied

Samples from	Natural Water Content (%)	Bulk Unit Weight (g/cm^3)	Dry Unit Weight (g/cm^3)
Block # 01	28.4	1.79	1.39
	30.2	1.84	1.41
	29.8	1.81	1.39
	34.3	1.73	1.29
Block # 02	55.3	1.90	1.22
	52.1	1.83	1.20
	50.2	1.75	1.16
	51.9	1.79	1.17
Block # 03	52.0	1.90	1.25
	52.6	1.95	1.28
	51.7	1.88	1.24
	51.1	1.81	1.20
Block # 04	59.7	1.78	1.14
	58.6	1.77	1.11
	57.9	1.95	1.23
	58.0	1.92	1.21

carried out according to ASTM D-427 method.

The results of the Atterberg limits for 16 samples from the four different soil blocks are summarized in Table 3.2. Referring to Tables 2.2 & 2.3 in Chapter 2 where Gerald in 1974 and Tadanier in 1984 have classified expansive soils based on their index properties, Al-Qatif clays can be characterized as highly expansive since the values of Atterberg limits for all samples fall within the range of very high degree of expansion.

3.2.4 Unified Soil Classification

The liquid limit and plasticity index are plotted in plasticity charts shown in Figs. 3.8 to 3.11. The charts show that all data points lie just above A-line and to the right of L.L. = 50% line. Consequently, expansive soils studied, belong mainly to the high plasticity inorganic clay groups (CH in the Unified Soil Classification System, USCS) confirming the range of values obtained for other expansive soils by Tadanier (1984), Gupta et al (1967) and test results by Borchardt (1977) on soils responsible for landslides. Most of the data points in the plasticity chart are very close to A-line indicating the presence of inorganic silts (MH in USCS). This fact is also confirmed by the grain size distribution curves for soil samples from the four different blocks which indicate that Al-Qatif soils contain 30-40% silt content.

From Table 3.2 and plasticity charts, it can be concluded that

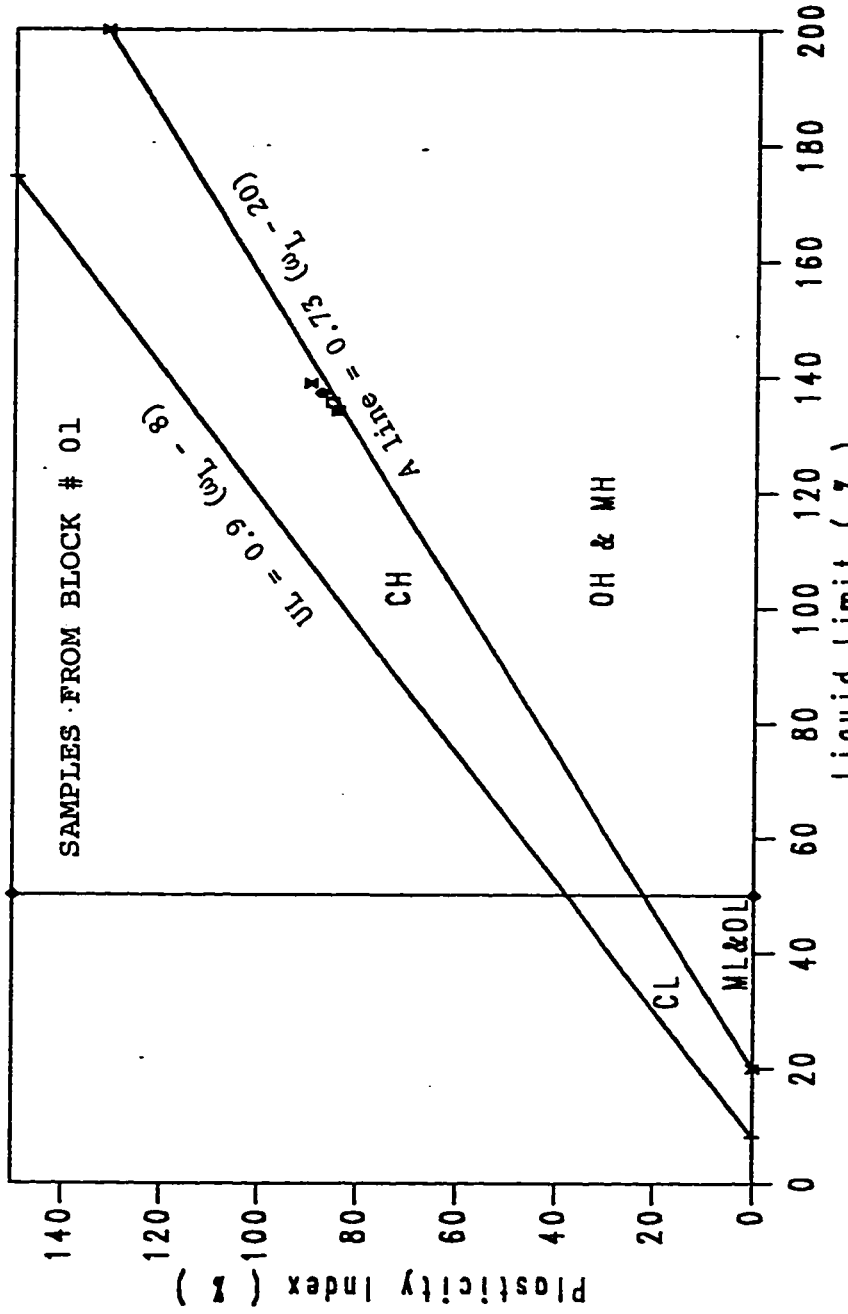


Fig. 3.8 : Plasticity Chart of Al-Qatif Clays from Test Pit # 01

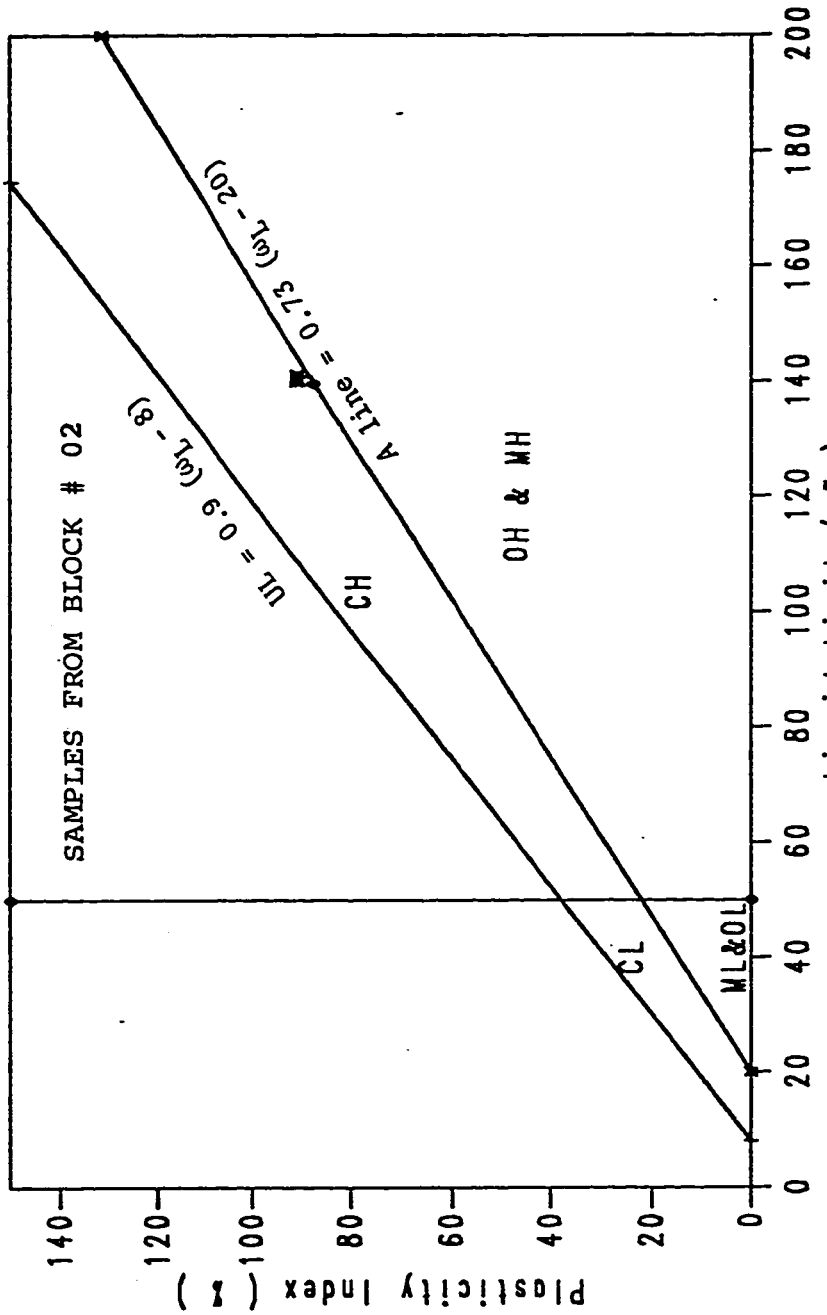


Fig. 3.9 : Plasticity Chart of Al-Qatif Clays from Test Pit # 01

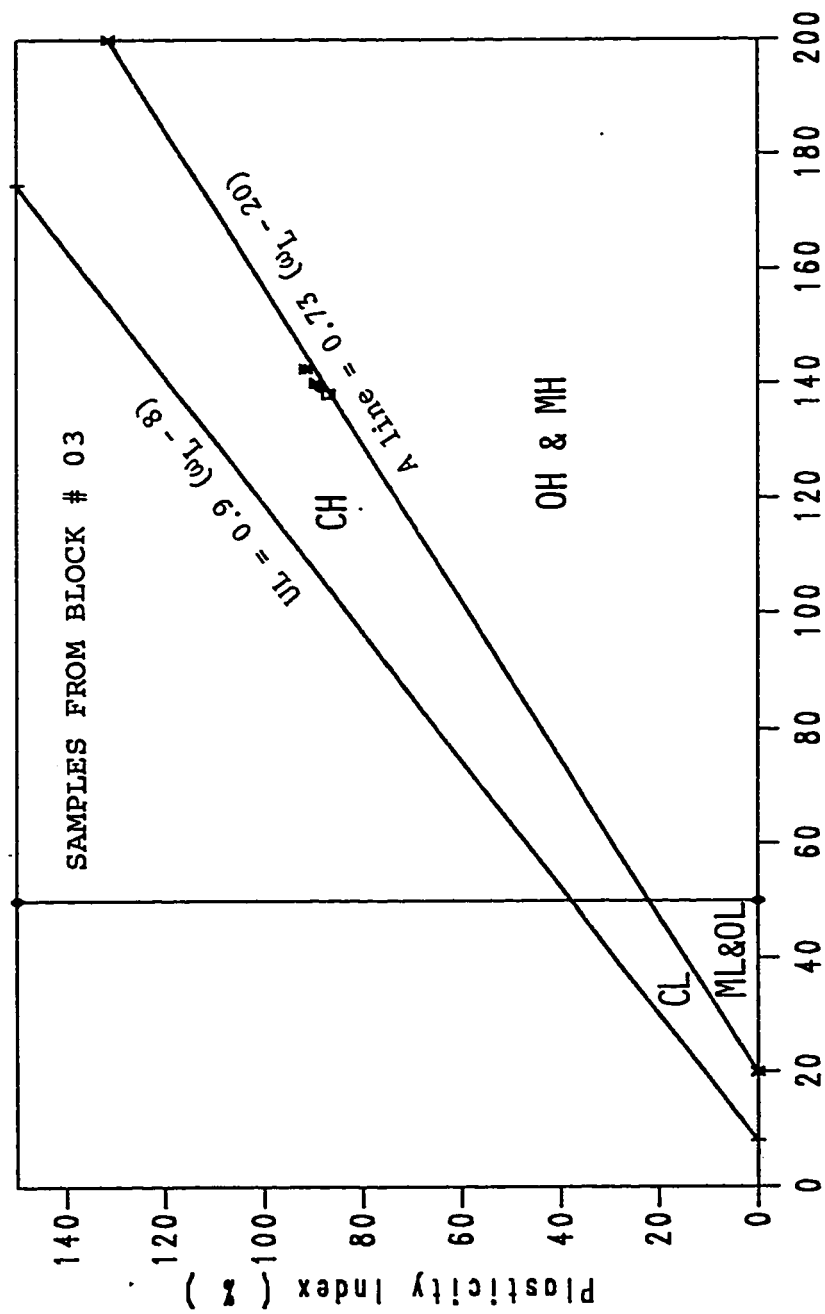


Fig. 3.10 : Plasticity Chart of Al-Qatif Clays from Test Pit # 02

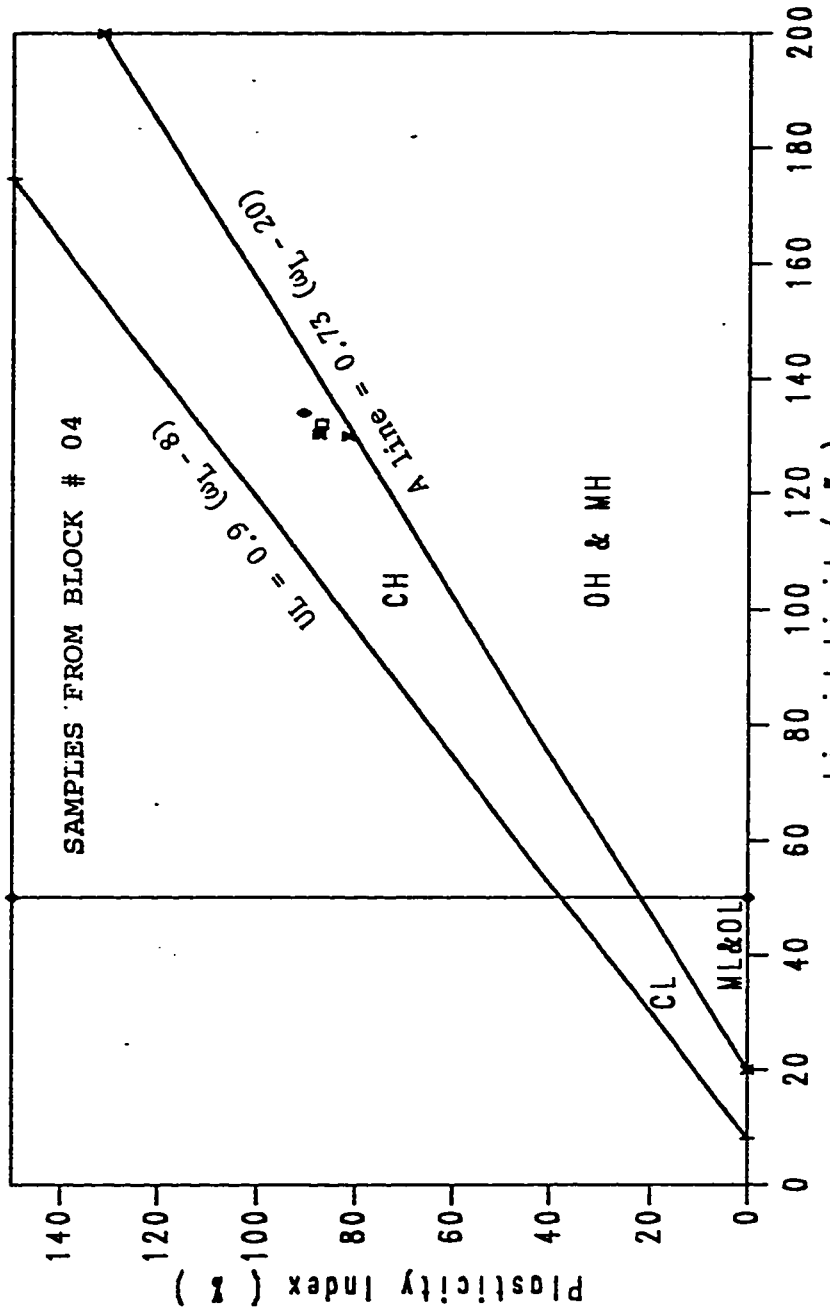


Fig. 3.11 : Plasticity Chart of Al-Qatif Clays from Test Pit # 03

Table 3.2 : Atterberg Limits of Al-Qatif Clay Samples

Samples from	L.L. (%)	P.L. (%)	P.I. (%)	S.L. (%)
Block # 01	135.6	50.2	85.4	10.5
	138.9	48.9	90.0	11.2
	134.1	49.8	84.3	9.9
	137.2	49.5	87.7	10.2
Block # 02	140.2	51.3	88.9	12.0
	141.2	50.4	90.8	12.1
	139.8	48.9	90.9	11.8
	139.6	52.1	87.5	11.7
Block # 03	138.0	51.2	86.8	9.8
	140.0	50.7	89.3	8.7
	142.5	51.1	91.4	6.9
	139.3	50.6	88.7	9.5
Block # 04	132.0	45.0	87.0	11.9
	129.8	48.2	81.6	12.5
	130.2	42.5	87.7	11.8
	133.9	43.0	90.9	11.1

the soil mass in the study area is homogeneous. However, for the same block, the values of Atterberg limits are different for samples taken from different sides of the block. The expansive characteristics are not reflected by Unified Soil Classification System. So, a classification system relating swelling characteristics with index properties has been expressed in the form of charts & empirical relationships by many investigators. Dakshinamurty & Raman (1977) have supplemented plasticity chart with swelling as shown in Fig. 3.12. Putting the data points in the chart, Al-Qatif clays can be characterized as extra high swelling [4].

3.3 Consolidation Tests

The general procedure for laboratory evaluation of consolidation characteristics of clays involves the classical one dimensional test. Soil sample is placed inside a metal ring which allows water flow and soil movement only in the vertical direction. In the field some lateral water movement may occur, as well as some slight lateral soil movement. Neither of these effects is probably very important when considering the overall settlements due to consolidation based on extrapolating laboratory results to the field [28].

The apparatus used to perform consolidation test is shown in Fig. 3.13. It consists of a consolidometer, a metal ring (6.9 cm x 1.9 cm) and a deformation dial gauge. Loads were applied through a dead weight system attached to the lever arm. The porous stones at the top and bottom of the sample provide a double drainage for faster

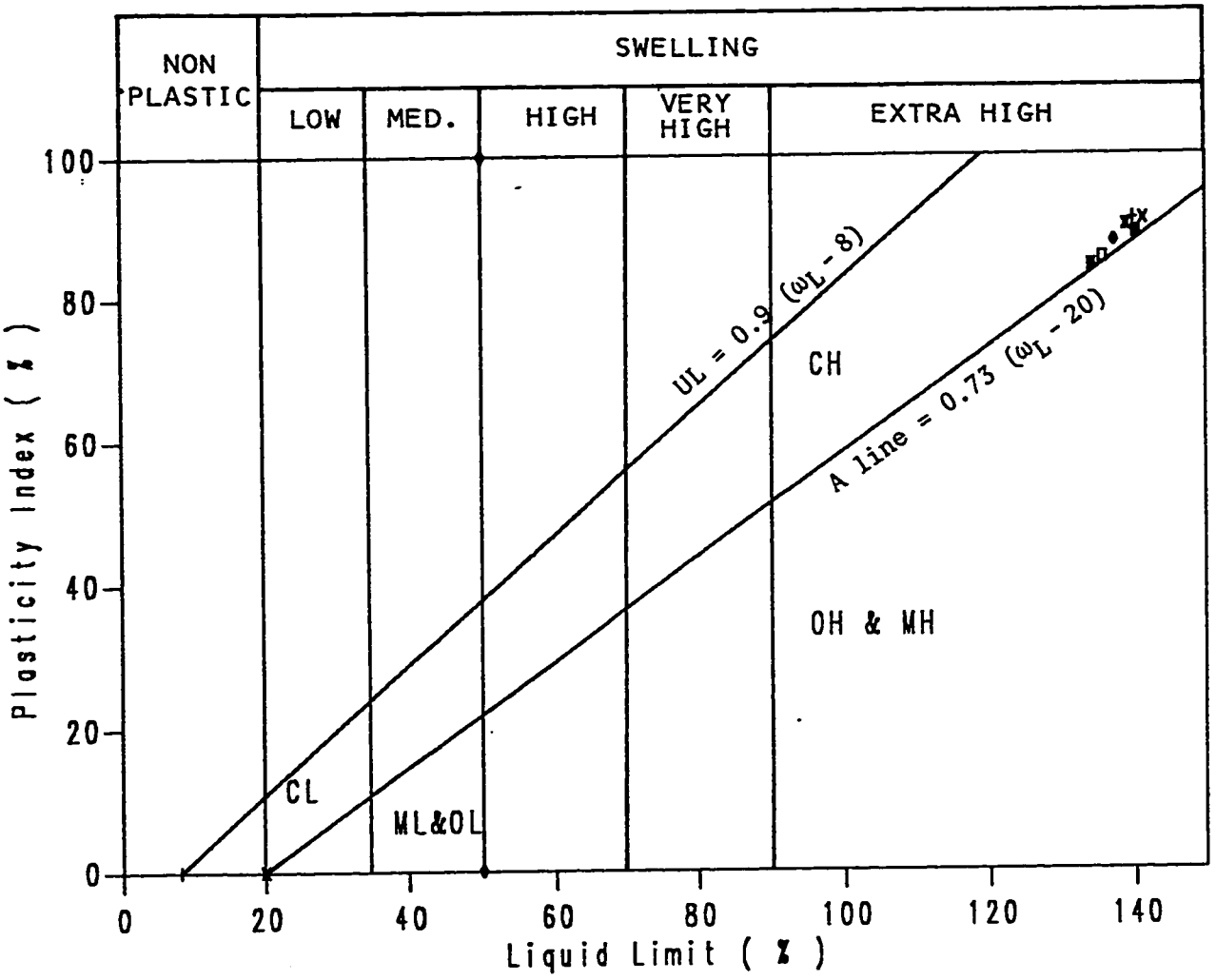


Fig. 3.12 : Classification of Al-Qatif Clays according to Dakshinamurthy & Raman Chart (Ref. 4)

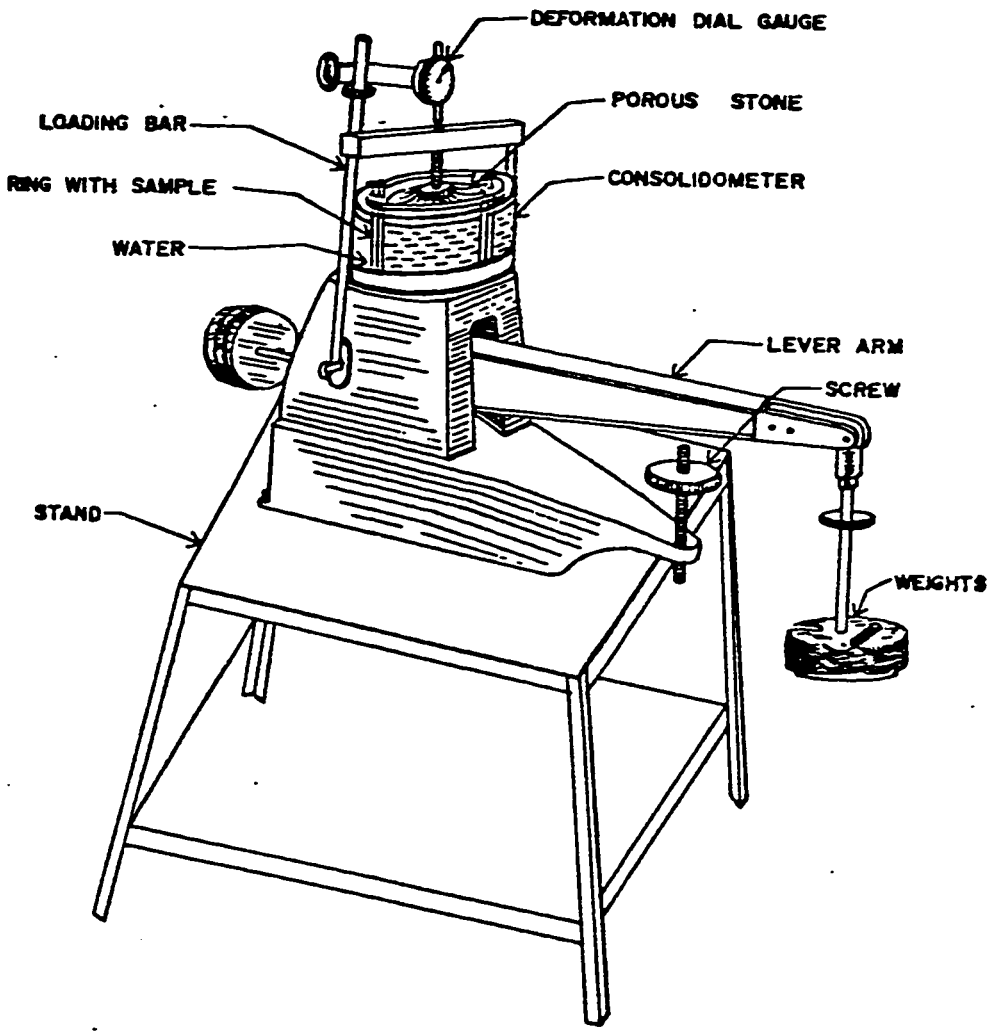


Fig. 3.13 : Consolidation Test Setup (Conventional 1-D Consolidometer)

compression. ASTM D-4186 procedure was followed. The laboratory data are used for the determination of important coefficients related to compression & swelling. One of the important coefficients is the swell index, C_s, which is commonly used to measure expansion [17].

$$\text{Swell index } C_s = \frac{-\Delta e}{\Delta \log \bar{p}} \qquad \text{(Unloading)}$$

where Δe is the change in the void ratio and $\bar{p}$ is the effective consolidation pressure.

The compression index C_c is a measure of compressibility of the sample. The compressibility is defined as the "property of a soil or rock pertaining to its susceptibility to decrease in volume when subjected to load" (ASTM, 1983). The compression index, C_c, can be obtained from the slope of the linear portion of the virgin compression curve on the graphs of void ratio versus logarithm of effective pressure. The relationship is expressed as:

$$\text{Compression index } C_c = \frac{-\Delta e}{\Delta \log \bar{p}} \qquad \text{(loading)}$$

C_s for unloading curve, is always much smaller than C_c for virgin compression. In most of the cases, expansion curves tend to be parallel. Thus C_s is more or less same for all values of $\bar{p}$ [17].

Samples were extracted from the four blocks and were subjected to conventional consolidation test. Figs. 3.14 to 3.17 show the

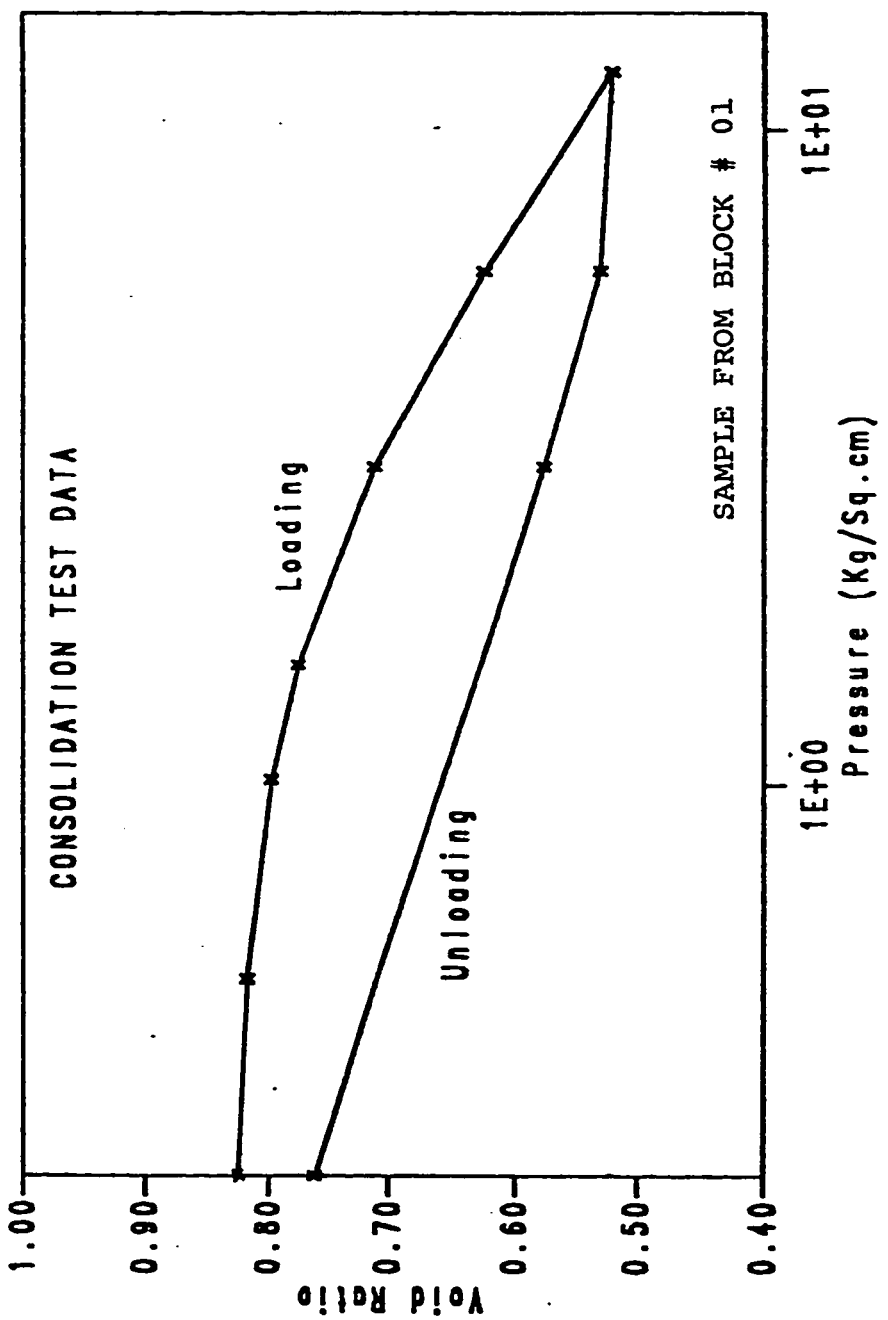


Fig. 3.14 : Plot of e vs log $\bar{P}$ for Sample from Block # 01

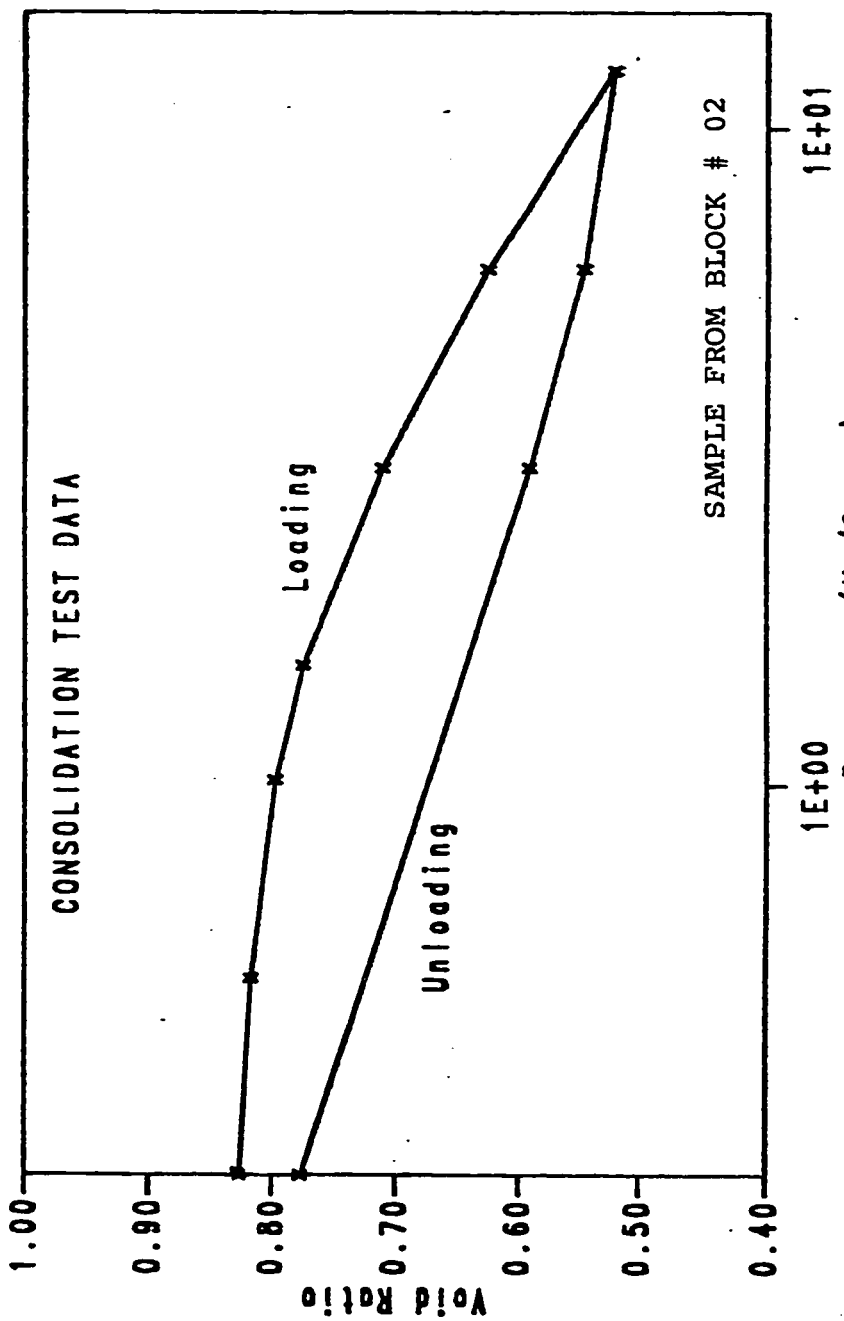


Fig. 3.15 : Plot of e vs log $\bar{P}$ for Sample from Block # 02

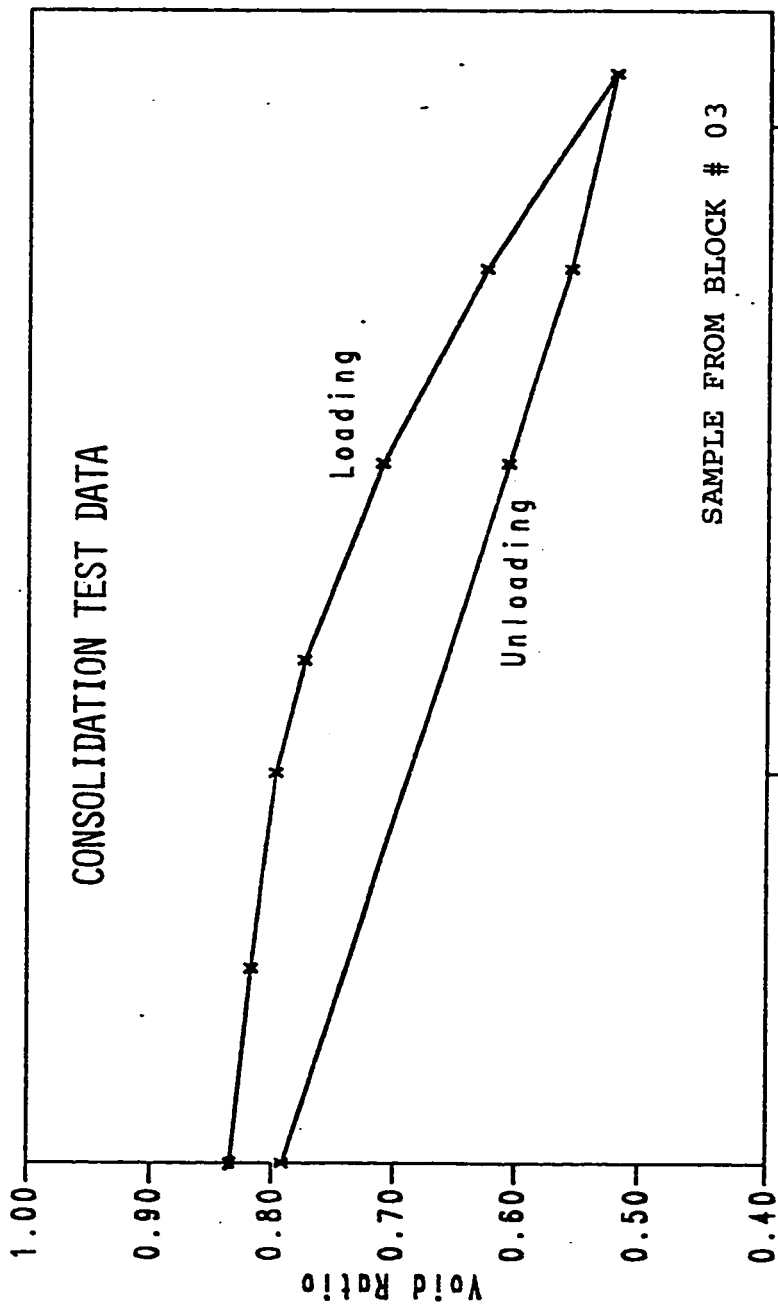


Fig. 3.16 : Plot of e vs log $\bar{P}$ for Sample from Block # 03

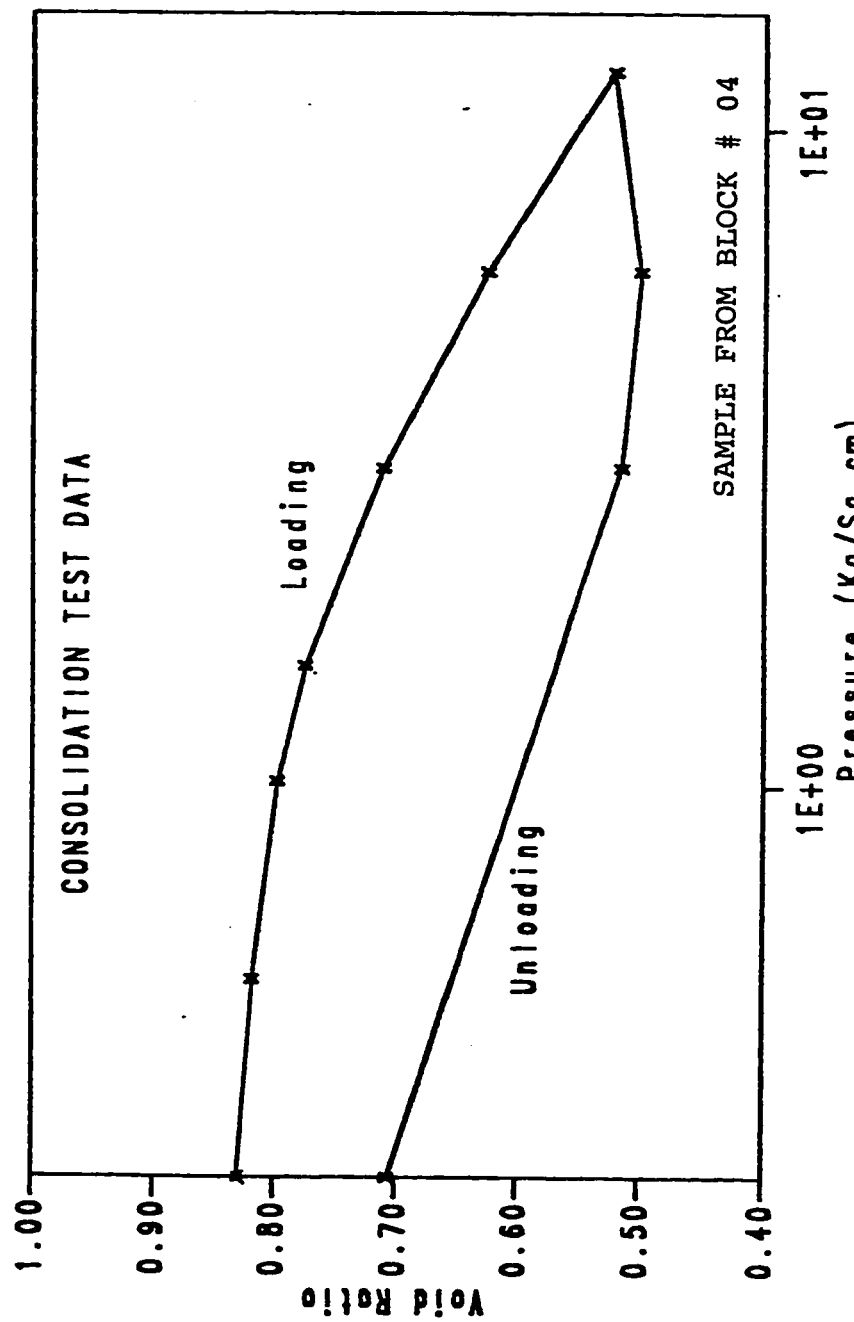


Fig. 3.17 : Plot of e vs log $\bar{P}$ for Sample from Block # 04

one dimensional compression curves for soil samples studied. The sample in the consolidometer was subjected to prescribed load increments of 25, 50, 100, 200, 400, 800, 1600 kPa. The sample was then unloaded in three steps. It is clear that sample from block # 03 swell the most since it reached very near to its original voids ratio on unloading while the swelling of soil sample from block # 04 was the least. This observation can be verified from the values of swell index C_s shown in Table 3.3, where the sample from block # 03 possesses the highest swell index of 0.173. Comparing the values of swell index of Al-Qatif soils with those of natural soils investigated by other researchers, Table 3.4, it can be concluded that the Al-Qatif soils are quite expansive in nature.

The results of the consolidation test drawn in Figs. 3.14 to 3.17 show that the compression curves consist of two parts. The initial portion with flat slope represents the recompression of soil skeleton, while the second portion which has steeper slope represents the virgin compression of soil. The swelling curves of the soil samples studied are straight lines showing abrupt increase in volume after unloading the specimen. The slope of these lines is about the same for all samples.

3.4 Moisture Density Relationships

Artificially compacted expansive clays sometimes do not remain compacted, but expand upon entry of water. Sometimes the action is

Table 3.3 : Consolidation Test Results of Al-Qatif Clays

Sample from	Initial Voids Ratio (e_o)	Compression Index (c_c)	Swell Index (c_s)
Block # 01	0.834	0.625	0.170
Block # 02	0.836	0.624	0.172
Block # 03	0.841	0.624	0.173
Block # 04	0.835	0.625	0.170

Table 3.4 : Compression and Swell of Some Natural Soils (Bolt, 1958)

Soil	L.L.	P.L.	Compression Index (c_c)	Swell Index (c_s)	Reference
Expansive Soil A	84	48	-	0.25	Dawson, 1957
Expansive Soil B	87	42	0.21	0.15	Dawson, 1957
Boston Blue Clay	41	20	0.35	0.09	Mitchell, 1956
Chicago Clay	58	21	0.42	0.12	Mitchell, 1956

scarcely detectable because uplift is uniform, but other times it causes major problems, particularly under buildings or pavements. Reducing the compactive effort to give a lower initial density is not an effective control on expansion, since the clay lumps still comprise an expanding skeleton. Furthermore there is a disadvantage in loose compaction because the soil is much more susceptible to daily fluctuations in water table.

Where expansion is to be avoided, clay may be compacted very wet preferably above the optimum moisture content. In cases where such a high moisture content causes the soil to be too weak - a common practice is to mix in several percent of hydrated lime, which reacts with the clay minerals, greatly reducing their expansive character and allowing compaction at an optimum moisture content for maximum dry density.

Standard Proctor test was performed to obtain moisture-density relationships for soil samples from the four blocks. Here the soil was compacted in a cylindrical mold 10.16 cm in diameter and about 11.6 cm high. The soil was placed in the mold in three equal layers, and each layer was compacted by 25 blows of a 2.5 kg metal rammer having a sticking face of 5.08 cm in diameter. The rammer was allowed to fall freely through a height of 30.5 cm for each blow. The amount of energy thus applied was established by R.R. Proctor, as the amount which would give a maximum density in the laboratory test approximately equal to that which it is feasible to obtain in ordinary

field compaction operations. The maximum dry density and optimum moisture content for the samples tested are given in Table 3.5. The compaction curves and zero air void (ZAV) curves are being plotted in Figs. 3.18 through 3.21 for the different samples. The natural dry density was found to be within the range of 1.1 - 1.4 g/cm^3, and the average moisture content varied from 28% to 60% as summarized in Table 3.1. The average maximum dry density obtained from compaction curves, Fig. 3.17, was about 1.193 g/cm^3 at an average moisture content of 36.6%. Comparing the values of average maximum dry density $\gamma_{d(max)}$ and corresponding optimum moisture content $w_{opt.}$ with the average natural dry unit weight and natural water content in Table 3.1, it was observed that the dry density can be increased by 50% after compaction. All soil samples except samples from block # 01 were possessing natural water content above optimum moisture content.

Table 3.5 : Compaction Test Results for Al-Qatif Soils

Sample from	Max. Dry Density γ_{max} (g/c.c)	Optimum Moisture Content $w_{opt.}$ (%)
Block # 01	1.195	32.5
Block # 02	1.194	35.0
Block # 03	1.199	38.9
Block # 04	1.186	40.0

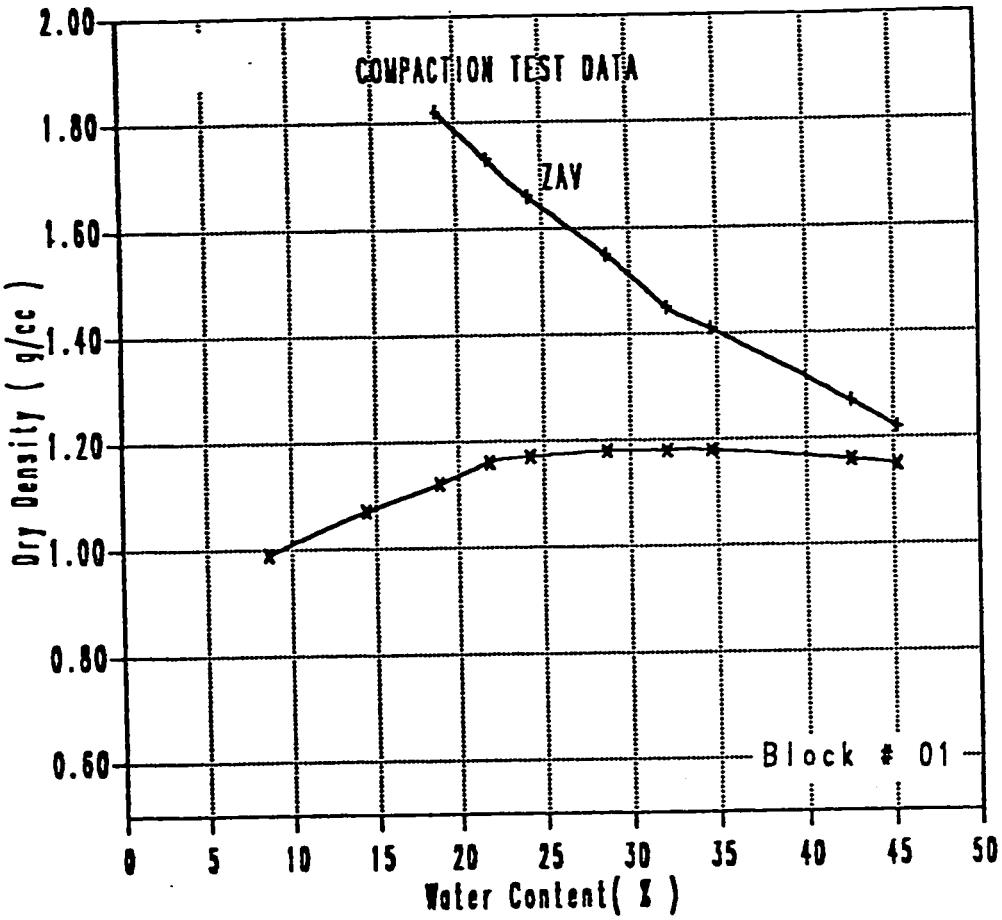


Fig. 3.18 : Moisture-Density Relationships for Sample from Block # 01

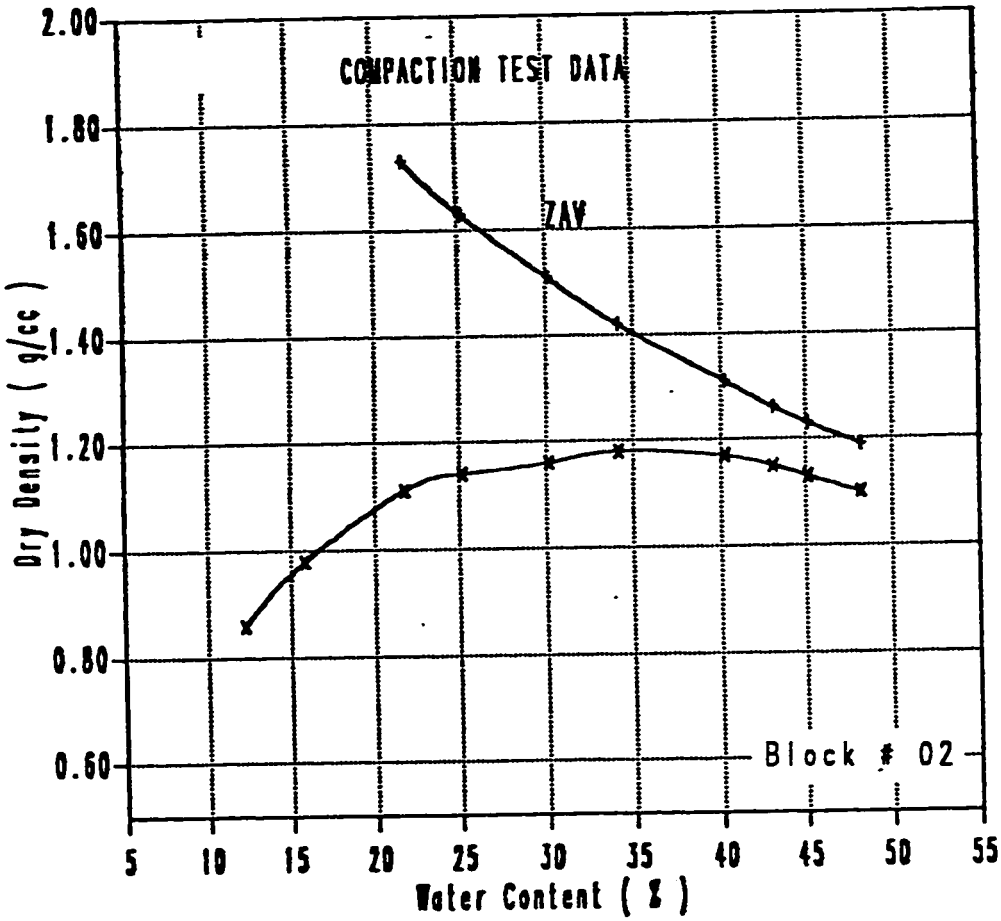


Fig. 3.19 : Moisture-Density Relationships for Sample from Block # 02

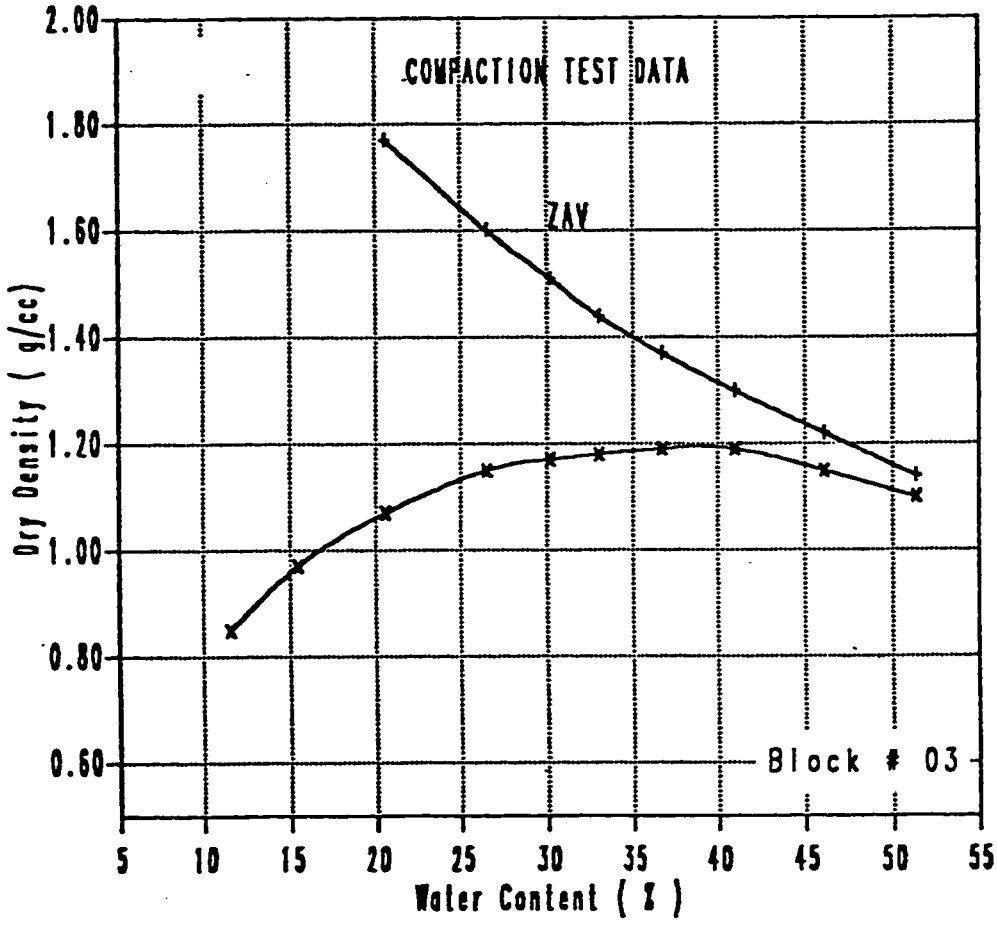


Fig. 3.20 : Moisture-Density Relationships for Sample from Block # 03

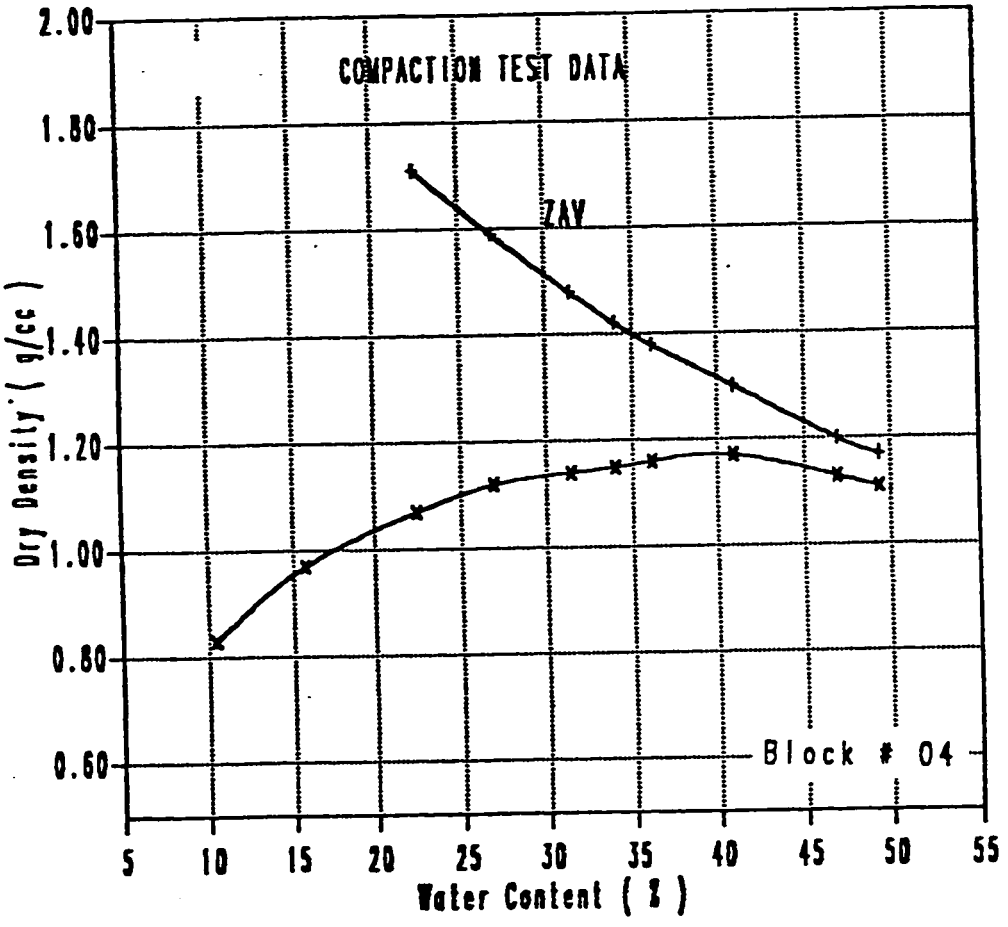


Fig. 3.21 : Moisture-Density Relationships for Sample from Block # 04

CHAPTER 4

MINERALOGICAL IDENTIFICATION

4.1 Introduction

A knowledge of the mineralogical and chemical composition, and organic contents can be of great value for understanding engineering properties of soil. The general physical characteristics of the important clay minerals are reasonably well known, and influences of such constituents as organic matter and the non-crystalline oxides of iron, silicon, and aluminium are becoming better understood. Quantitative estimation of minerals in a soil sample can ordinarily be made by using x-ray diffraction. Simple chemical tests can be used to indicate the presence of organic matter and other constituents. For determination of soil fabric, latest technology called the Scanning Electron Microscopy (SEM) has proved to be very useful. Further information on the sample can be obtained from chemical methods such as cation exchange capacity or chemical analysis of ions. The methods and techniques that may be employed for mineralogical composition and study of soil fabric are as follows [1]:

1) Particle size analysis and separation.
2) Various pretreatments prior to mineralogical analysis.
3) Chemical analysis for determination of free oxides, hydroxides, amorphous constituents, and organic matter.
4) Petrographic microscopic study of silt and sand grains.

5) Scanning Electron microscopic study for soil fabric.

6) XRD for qualitative and quantitative estimation of minerals.

7) Differential Thermal Analysis for identification of components (clay and non-clay minerals).

8) Determination of specific surface.

9) Chemical analysis for cation exchange capacity, exchangeable cations, pH and soluble salts.

10) Staining test for identification of clay minerals.

4.2 Mineralogical Composition by X-ray Diffraction

X-ray diffraction is the most widely used method for identification of clay minerals and to study the structure of crystals. When x-rays strike a crystal, they may penetrate to a depth of several million layers before being absorbed. At each atomic plane a minute portion of the beam is absorbed by individual atoms that can oscillate as dipoles and radiate in all directions. Radiation in certain directions will be in phase and can be interpreted in simplistic fashion as a wave resulting from a reflection of the incident beam. In-phase radiations emerge as a coherent beam and can be detected on film or by a radiation counting device. The orientation of parallel atomic planes, relative to the direction of the incident beam, at which radiations are in phase depends on the wave length of the x-rays and the spacing between the atomic planes [11].

Fig. 4.1 shows a parallel beam of x-rays of wave length λ striking a crystal at an angle θ to parallel atomic planes spaced at

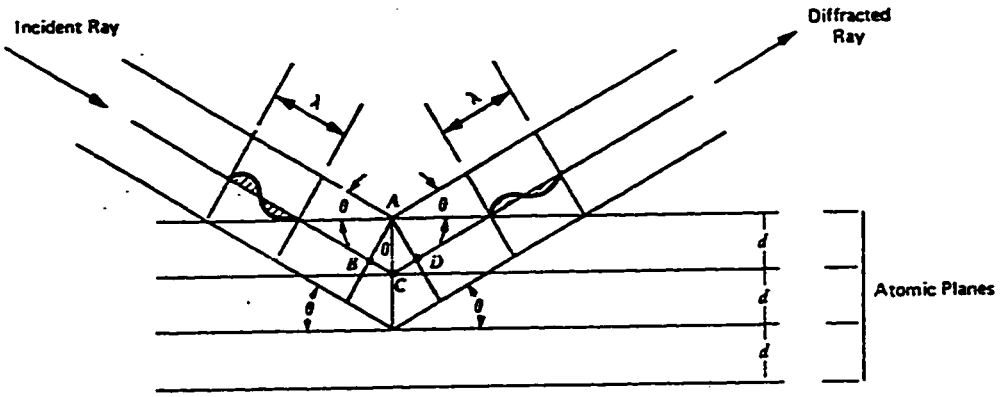


Fig. 4.1 : Geometry of Diffraction Pattern according to Bragg's Law

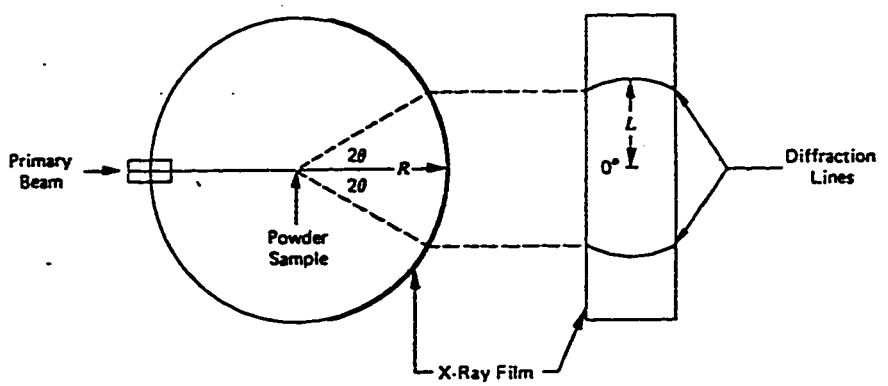


Fig. 4.2 : Schematic Arrangement of an X-ray Diffractometer

distance d. If the reflected wave from C is to reinforce the wave reflected from A, then the path differences between the two waves must be an integral number of wave lengths $n\lambda$. From Fig. 4.1, the difference in path lengths is the distance BC + CD. Thus,

$$BC + CD = n\lambda$$

But from symmetry BC = CD and by trigonometry CD = d sin 0. Thus the necessary condition is given by Bragg's law, as:

$$n\lambda = 2d \sin \theta \tag{4.1}$$

This law forms the basis for identification of crystals using x-ray diffraction. Since no two minerals have the same spacings of interatomic planes in three dimensions, the angles at which diffraction occurs, and the atomic spacings calculated therefrom are used for identification of these minerals in a clay sample. X-ray diffraction is well suited for identification of the clay minerals because the (001) spacing is the characteristic for each clay mineral group. The common non-clay minerals occurring in soils are also detectable by x-ray diffraction [9].

Because the small size of most soil particles prevents the study of single crystals, the powder method was used. In the powder method, a small sample having particles of all possible orientations is placed in a beam of parallel x-rays and diffracted beam of various intensities are either recorded on a film strip placed around the periphery of a circular camera as shown in Fig. 4.2, or scanned by a

Geiger-Muller counting tube and read directly or recorded automatically. The powder method is preferred because the very large number of particles in a powder sample insures that some will be properly oriented to produce a reflection.

The complete x-ray diffraction pattern, either film or strip chart record, will consist of a series of reflections of different intensities and values of 0. Each reflection must be accounted for in terms of some components of the sample. The first step in the analysis is to determine all values of d/n for the particular type of radiation (which determines λ) using equation (4.1). The test pattern may be compared directly with patterns for known materials [1].

4.3 XRD Analysis of Al-Qatif Clays

Soil samples collected from Al-QAtif were examined in the Materials Characterisation Laboratories (MCL), Research Institute at KFUPM for identification of clay and non-clay minerals. Analysis on XRD was carried out to determine the phases (minerals) present in the samples.

The samples were dried and were reduced to powdered form. A powder diffraction pattern was established by a vertical goniometer on the x-ray diffractometer. The operating conditions of XRD analysis were:

Cu broad focus tube at 45 KV and 30 ma.

Auto divergence slit and receiving slit = 0.2 ma.

Monochromator used and spinner on.

Speed and data collection was 0.01 degree 2θ/sec.

Angle recorded = 5 to 80 degree 2θ

The diffraction patterns shown in Figs. 4.3 to 4.6, were matched with the patterns of standards prepared by the Joint Committee of Powder Diffraction Analysis (JCPDA). This way the different clay and non-clay minerals present were identified.

Most of the specimens studied were found to be mixtures of clay and non-clay minerals. All samples contained quartz, atapulgite (polygorskite) and montmorillonite. The expansive nature can be attributed to the considerable amount of the montmorillonite present in most of the soil samples. The smectite (montmorillonite) has the greatest capacity for expansion due to intra-crystalline swelling or expansion of crystal lattice under wet conditions.

The summary of the results is given in Table 4.1. The numbers in the table represent approximate weight fraction of the phase in percentage. The weight fraction is calculated by comparing the intensity of the most intense peak of the phase with the JCPDA Standards. The intensities of the diffraction peaks are mainly governed by the amount of material, however lots of other factors also play an important role. Thus the weight fractions are only approximate.

Among the clay minerals, montmorillonite and polygorskite were present in all the samples. From Table 4.1 it can be concluded that

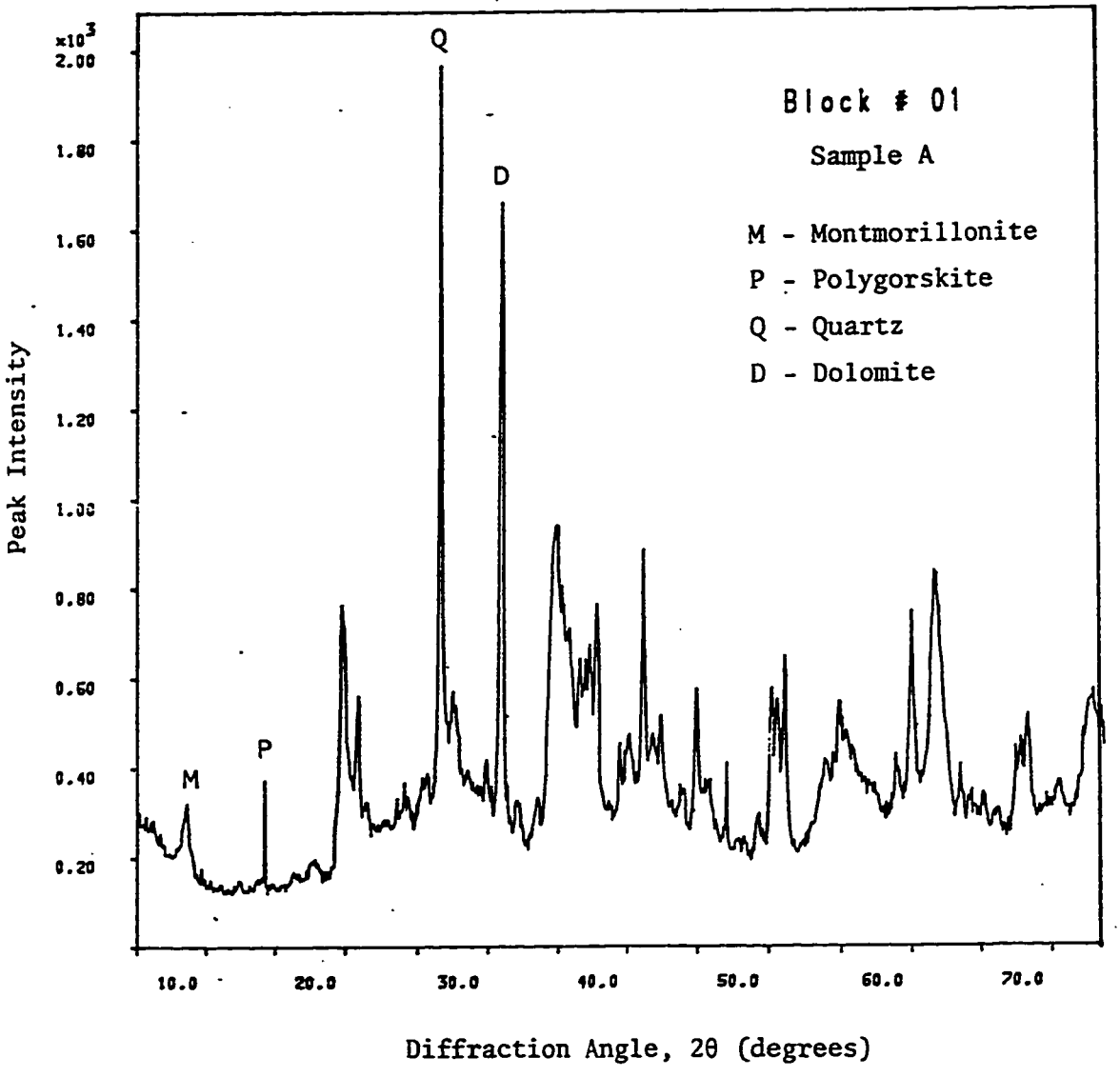


Fig. 4.3 : X-ray Diffractometer Chart for Al-Qatif Soil Sample. Peaks occur at specific 2θ angles converted to d spacings by Bragg's law.

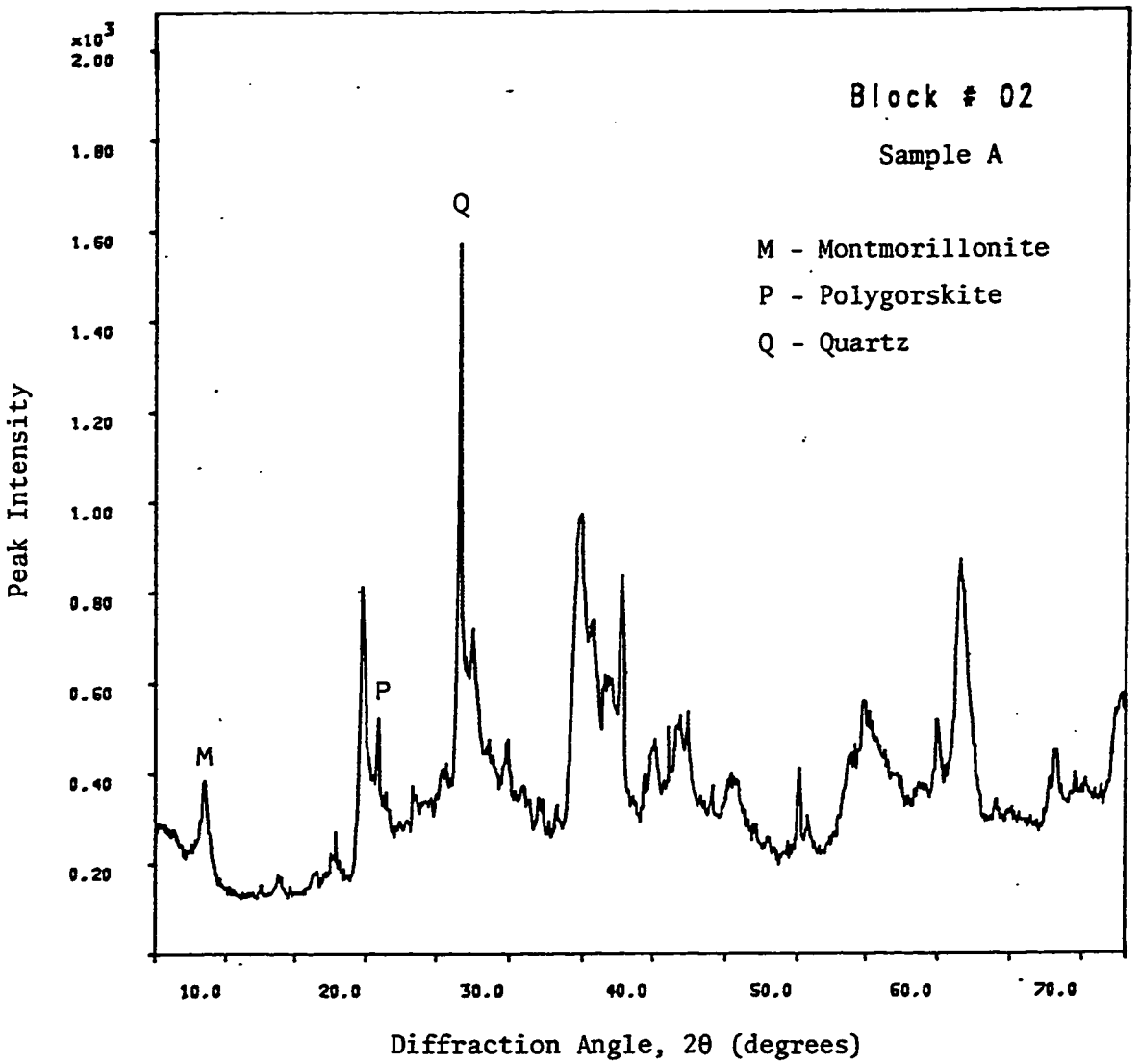


Fig. 4.4 : X-ray Diffractometer Chart for Al-Qatif Soil Sample. Peaks occur at specific 2θ angles converted to d spacings by Bragg's law.

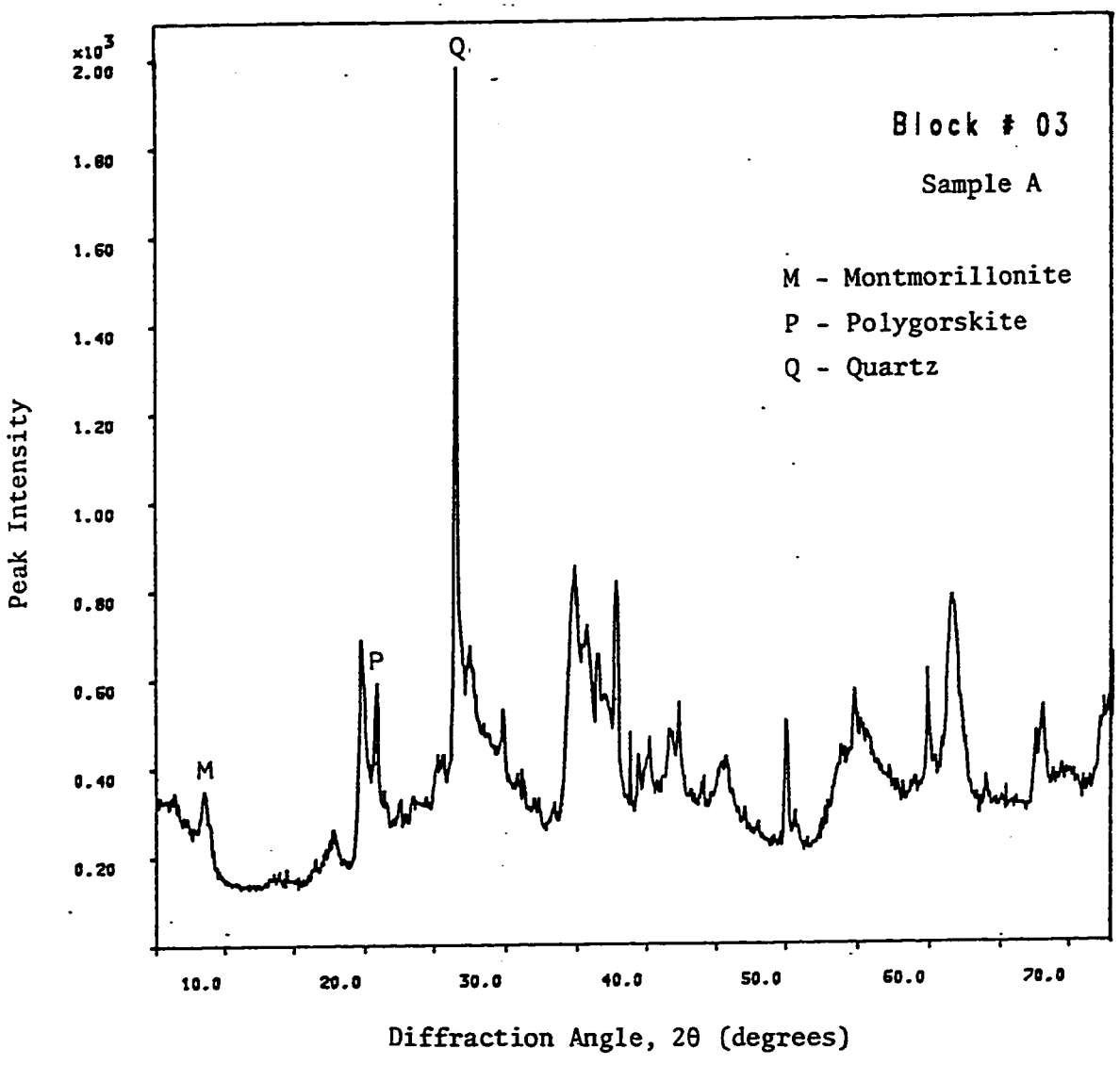


Fig. 4.5 : X-ray Diffractometer Chart for Al-Qatif Soil Sample. Peaks occur at specific 2θ angles converted to d spacings by Bragg's law.

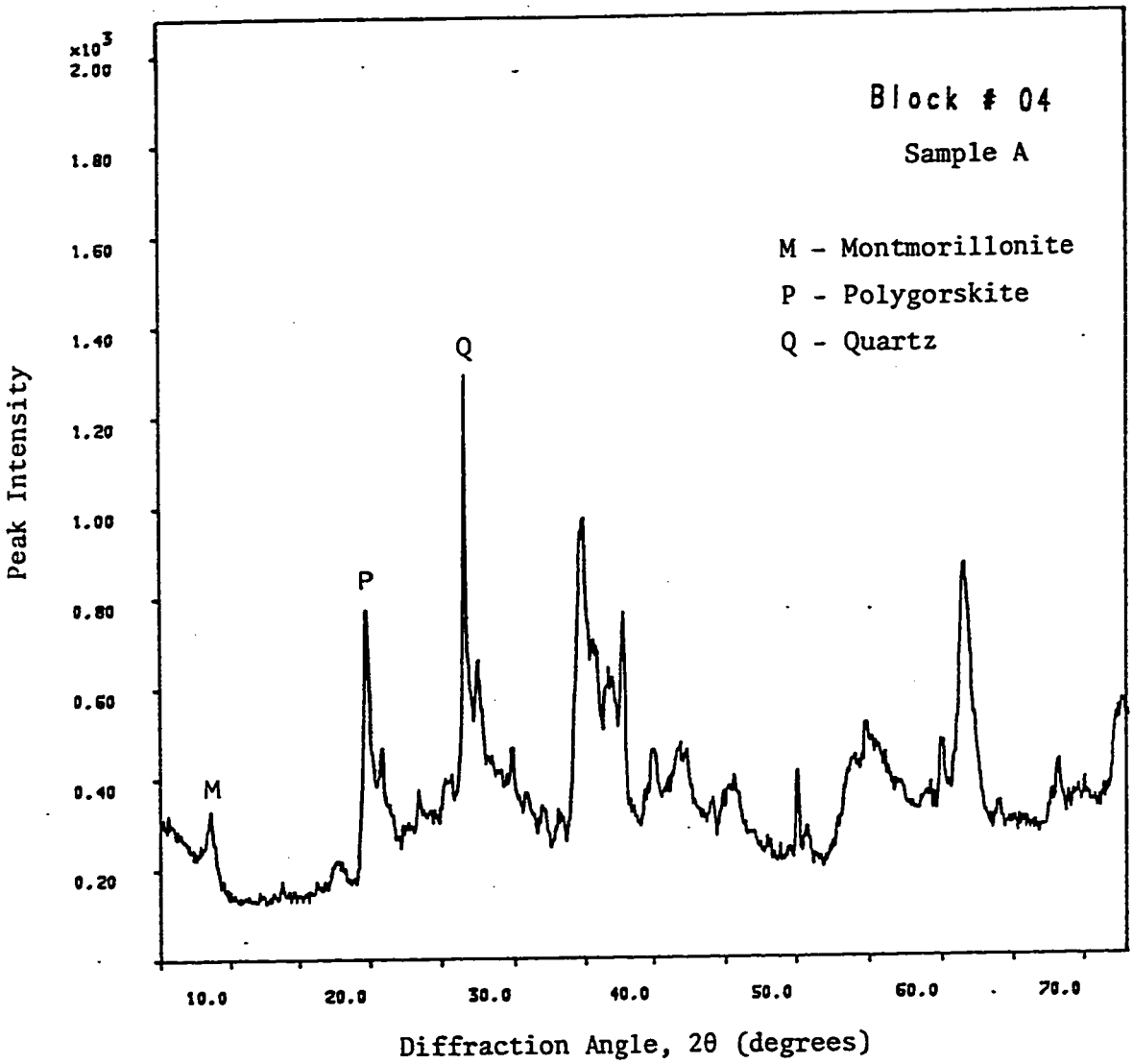


Fig. 4.6 : X-ray Diffractometer Chart for Al-Qatif Soil Sample. Peaks occur at specific 2θ angles converted to d spacings by Bragg's law.

Table 4.1 : Minerals Present in Al-Qatif Soil Samples

Minerals	Sample A	Sample B	Sample C
SAMPLES FROM BLOCK # 01			
Quartz SiO_2	45%	45%	62%
Palygorskite $Mg_5Si_8O_{20}(OH)_2.8H_2O$	7%	7%	33%
Dolomite $CaMg(CO_3)_2$	43%	43%	-
Montmorillonite $Na_{0.3}(Al,Mg)_2Si_4O_{10}(OH)_2.xH_2O$	5%	5%	5%
SAMPLES FROM BLOCK # 02			
Quartz SiO_2	53%	53%	67%
Palygorskite $Mg_5Si_8O_{20}(OH)_2.8H_2O$	25%	27%	25%
Montmorillonite $Na_{0.3}(Al,Mg)_2Si_4O_{10}(OH)_2.xH_2O$	22%	20%	8%
SAMPLES FROM BLOCK # 03			
Quartz SiO_2	70%	70%	72%
Palygorskite $Mg_5Si_iO_{20}(OH)_2.8H_2O$	18%	15%	5%
Montmorillonite $Na_{0.3}(Al,Mg)_2Si_4O_{10}(OH)_2.xH_2O$	12%	15%	23%
SAMPLES FROM BLOCK # 04			
Quartz SiO_2	60%	60%	66%
Palygorskite $Mg_5Si_8O_{20}(OH)_2.8H_2O$	28%	29%	31%
Montmorillonite $Na_{0.3}(Al,Mg)_2Si_4O_{10}(OH)_2.xH_2O$	12%	11%	3%

Al-Qatif clays are heterogeneous in nature since the percentage of different minerals differ from point to point for soil samples taken even from the same block. Following is the description of these minerals as regards their chemical composition, diffraction pattern and identification.

Clay minerals in which the component layers are not tightly bounded by K^+ ions (mica) or Mg^{+2} ions (vermiculites), but contain water molecules in a similar situation, are grouped together as montmorillonites. These minerals are all extremely fine-grained, do not form macroscopic crystals, and swell on the addition of water or organic liquids. Such minerals are also called montmorillonoids, montmorins or smectites. They are hydrous aluminosilicates bearing various formulas, one of which is $Na_{0.3}(Al,Mg)_2Si_4O_{10}(OH)_2.xH_2O$. The expansive character of this group of minerals provides the basis for their identification. When air dried, these minerals have basal spacings of 12 to 15 Å. After treatment with ethylene glycol or glycerol, the smectites expand to a value of 17 to 18 Å. When oven dried, $d_{(001)}$ drops to about 10 Å as a result of the removal of interlayer water [1].

Polygorskite, pilolite, lassalite, and attapulgite are synonymous; the name polygorskite has precedence. Attapulgite, the clay from Attapulgus, Georgia, is the name used for polygorskite in the United States. The ideal formula for half unit cell of polygorskite is

$Mg_5Si_8O_{20}(OH)_2.8H_2O$. Polygorskite has a wide range of crystallinity. A diffractogram of well-crystallized polygorskite shows a number of d reflections between 7° and 44° 2θ (12.7 Å to 2.07 Å). Reflections of sufficient intensity to appear on diffractograms occur at 10.5, 4.5, 3.23, and 2.62 Å. Above 400°C, the 10.5 Å reflection is replaced by a broad band, and near 800°C the structure is completely destroyed. Polygorskite has been recorded as a hydrothermal alteration product of pyroxenes and amphiboles but mostly found in marine deposits of Miocene age. Polygorskite requires an alkaline chemical environment for survival [10].

Quartz and dolomite were the only non-clay minerals present in the soil samples studied. Quartz is composed of silica tetrahedra grouped in such a way as to form spirals, with all tetrahedral oxygens bonded to silicon. The tetrahedral structure has a high stability. In addition, the spiral grouping of tetrahedra produces a structure without cleavage planes. There are no weakly bonded ions in the structure and the mineral has a high hardness. Dolomites are carbonate minerals and occur as bulkly particles, shells, precipitates or in solution form [9].

4.4 Identification of Soil Fabric Using SEM

The Scanning Electron Microscope (SEM) represents a rather recent development. With this instrument, secondary electrons are emitted from a sample surface what appear to be three dimensional

images. The SEM has a magnification range of x 20 to x 150,000 and a depth field of some 300 times greater than that of the light microscope.

The study of soil fabric with the SEM requires an evacuated sample chamber, so wet soils cannot be studied directly. The fluid which is an aqueous solution, be removed from the specimen before placing it in the instrument. Six techniques have been applied for removal of pore fluid (Towy and Wong, 1973). These techniques are (1) oven drying, (2) air drying, (3) humidity drying, (4) substitution drying, (5) freeze drying, and (6) critical point drying. The first two techniques are straight-forward and self-explanatory. Humidity drying is the process of drying the specimen under controlled humidity. Substitution drying involves the replacement of the pore fluid in the soil by a liquid of low surface tension such as methanol, acetone, or isopentane prior to drying (Lohness and Demirel, 1978). The last two techniques are used by biologists. In general, air drying is the most widespread technique for low moisture and stiff soils, whereas soils having a fragile fabric can be dried by rapid freeze drying [10]. For removal of pore water from Al-Qatif soil samples air drying method was used.

After drying the soil specimen, it is necessary to expose an undisturbed surface for study by fracturing and/or peeling. In order to fracture the specimen, it is scored and then bent or pulled to create a tensile failure surface. Peeling involves applying an

adhesive material to the specimen surface and then stripping it off to remove disturbed surface particles of the specimen with the adhesive. It is also necessary to coat SEM sample surfaces with a conducting film to prevent surface charging and consequent loss of resolution. Gold placed in a very thin layer (200 to 300 A) in a vacuum evaporator is often used [9].

The basic components of the SEM are the lens system, electron gun, electron collector, visual and recording Cathod Ray Tubes (CRTs), and the electronics associated with them. The electron beam enters the specimen chamber and strikes the specimen at a single location. Within the interaction volume, both elastic and inelastic scattering occur, producing detectable signals from backscattered electrons, secondary electrons, absorbed electrons characteristic and continuum x-rays, and cathodic radiation. By measuring the magnitude of these signals with suitable detectors, a determination of certain properties of the specimen, e.g. local topography, composition, arrangement of grains, etc. can be made at the signal location of the electron beam impact. In order to study more than a single location, the beam must be moved from place to place by means of a scanning system, illustrated in Fig. 4.7 [10].

4.5 Analysis of Al-QAtif Clays by SEM

Clay samples from the four blocks were analysed in Materials Characterisation Laboratory (MCL), Research Institute at KFUPM. The objective of this analysis was to study the morphology and to

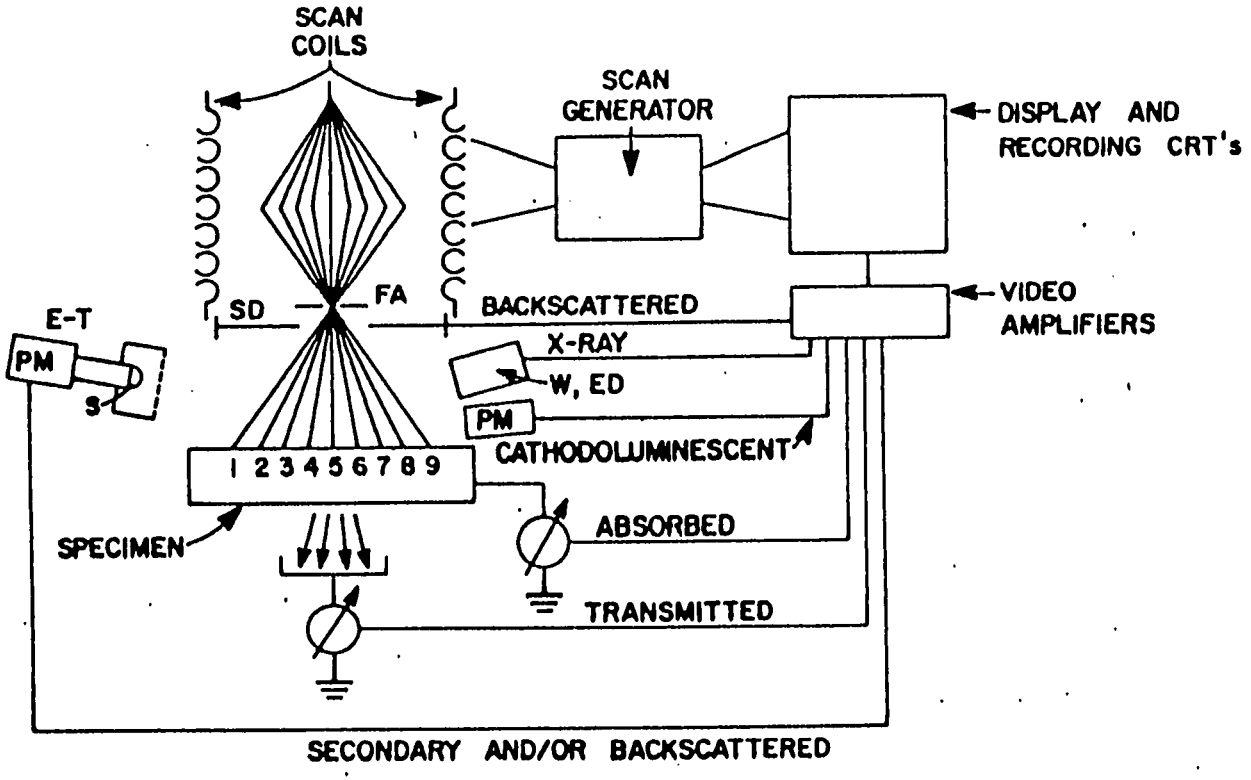


Fig. 4.7 : Schematic Illustration of Scanning System of the Scanning Electron Microscope.
Abbreviations: FA, final aperture; SD, solid state electron detector; ET, Everhart-Thornley detector; S, scintillator; PM, photomultiplier; W, wavelength-dispersive x-ray spectrometer; ED, energy-dispersive x-ray spectrometer; CRT, cathode ray tube. Numbers 1-9 indicate successive beam positions during a scanning sequence.

and to determine the geometry and particle arrangement present in the samples. Each sample was fractured into two different sections: parallel and perpendicular to layers. Thus there were eight samples to examine. All samples were then mounted on an aluminium sample holder and sputter coated with a very thin layer of gold. The SEM was operated at 20 KV accelerate voltage and the probe current was set at 6(-10) amp. Several photomicrographs, Figs. 4.8 to 4.11 and the corresponding EDS spectra, Figs. 4.12 to 4.15, were obtained for each sample to show the morphology of the different clay & non-clay minerals present in the samples.

In all the soil samples analyzed by SEM the dominating clay minerals were polygorskite and montmorillonite. The presence of montmorillonite is confirmed by SEM photomicrographs of the soil samples along the layers, showing clear flakes morphology. The flocculated clay particles are arranged in an arching skeleton enclosing large voids. In samples from block # 01 and # 04, Figs. 4.8(a) and 4.11(a), minerals are linked in edge-to-edge (flocculated) fashion while photomicrograph for sample from block # 02, Fig. 4.9(a) shows edge-to-face attachment of particles. Arrangement of particles in sample from block #03, Fig. 4.10(a) is different from all other samples. This arrangement has been termed as honeycomb (Lambe, 1958). The more or less continuous nebulous film-like assemblages confirm the presence of Na-montmorillonite. Ca-montmorillonite, which appear as irregular aggregates, were not observed in any of the samples.

Fig. 4.8(a)

Fig. 4.8(b)

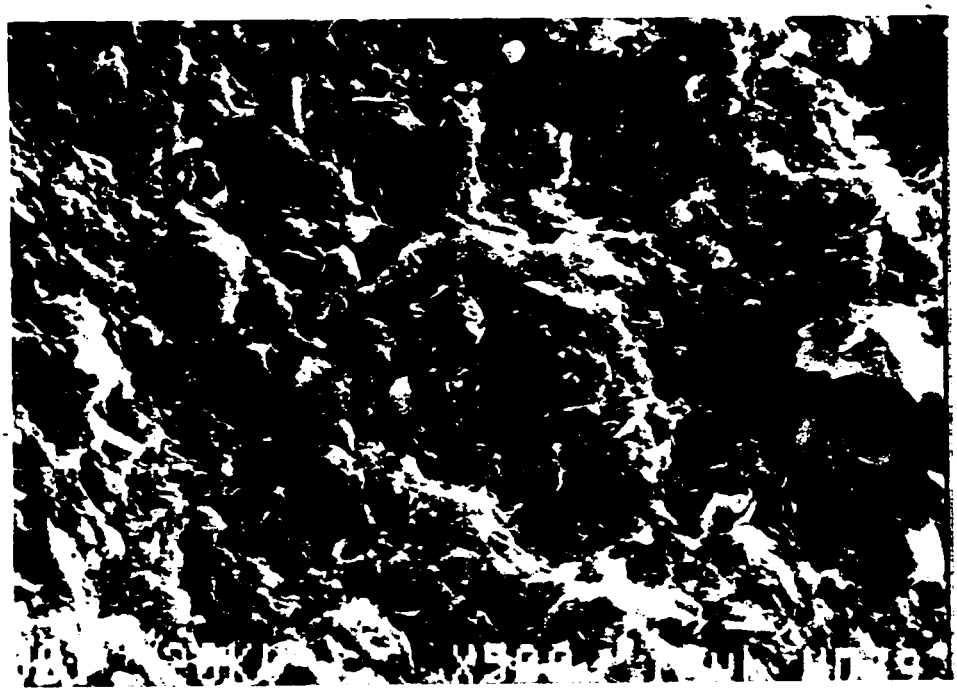


Fig. 4.8 : Scanning Electron Photomicrographs of Undisturbed Soil Fabrics for Samples from Block # 01. Upper photograph showing fabric along layers, lower one perpendicular to layers.

Fig. 4.9(a)

Fig. 4.9(b)

Fig. 4.9 : Scanning Electron Photomicrographs of Undisturbed Soil Fabrics for Samples from Block # 02. Upper photograph showing fabric along layers, lower one perpendicular to layers.

Fig. 4.10(a)

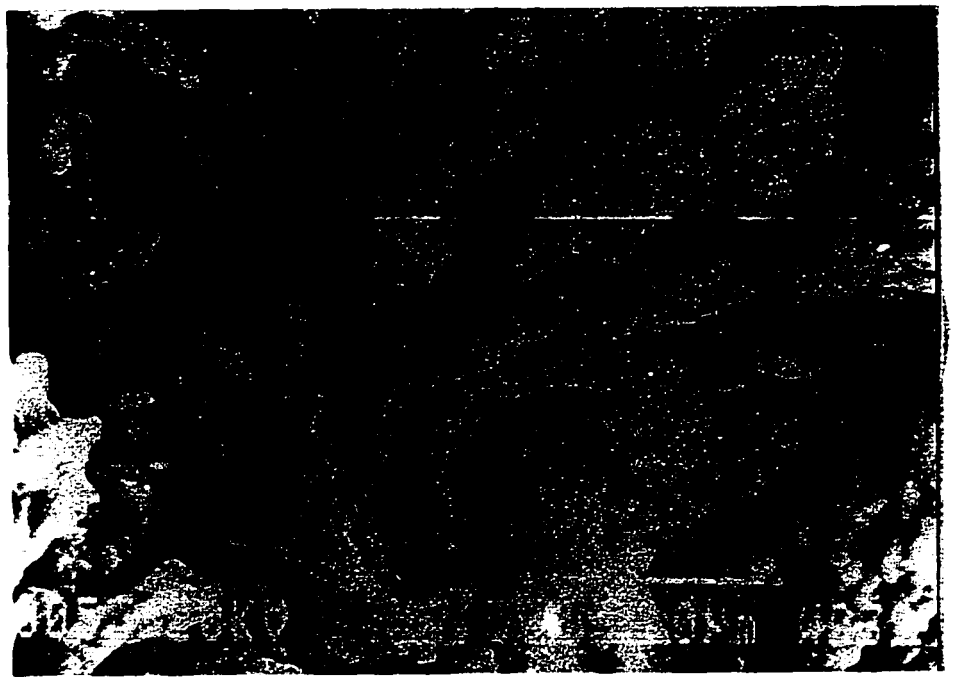


Fig. 4.10(b)

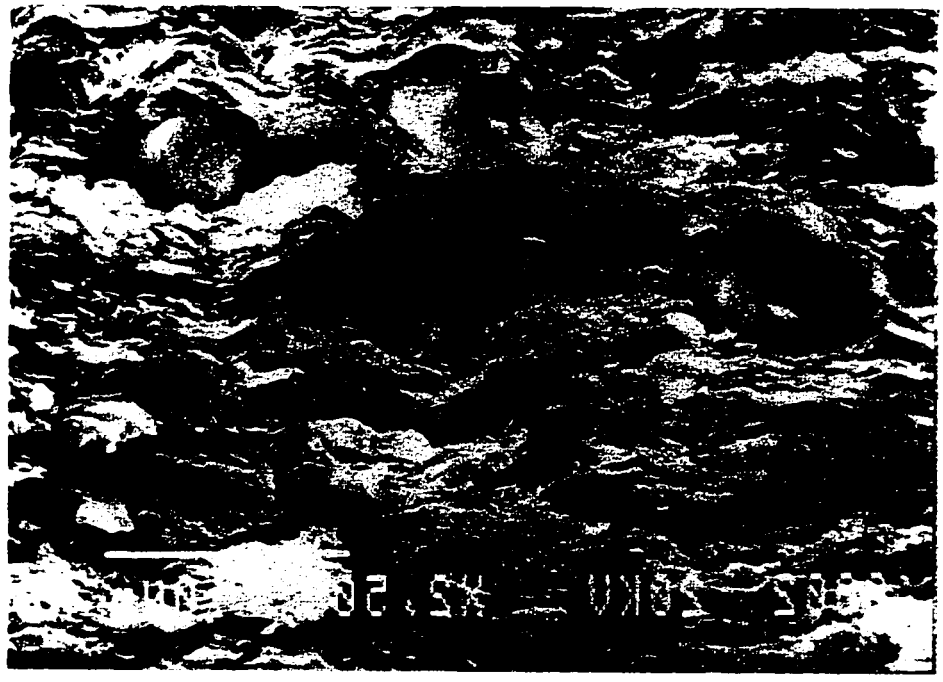


Fig. 4.10 : Scanning Electron Photomicrographs of Undisturbed Soil Fabrics for Samples from Block # 03. Upper photograph showing fabric along layers, lower one perpendicular to layers.

Fig. 4.11(a)

Fig. 4.11(b)

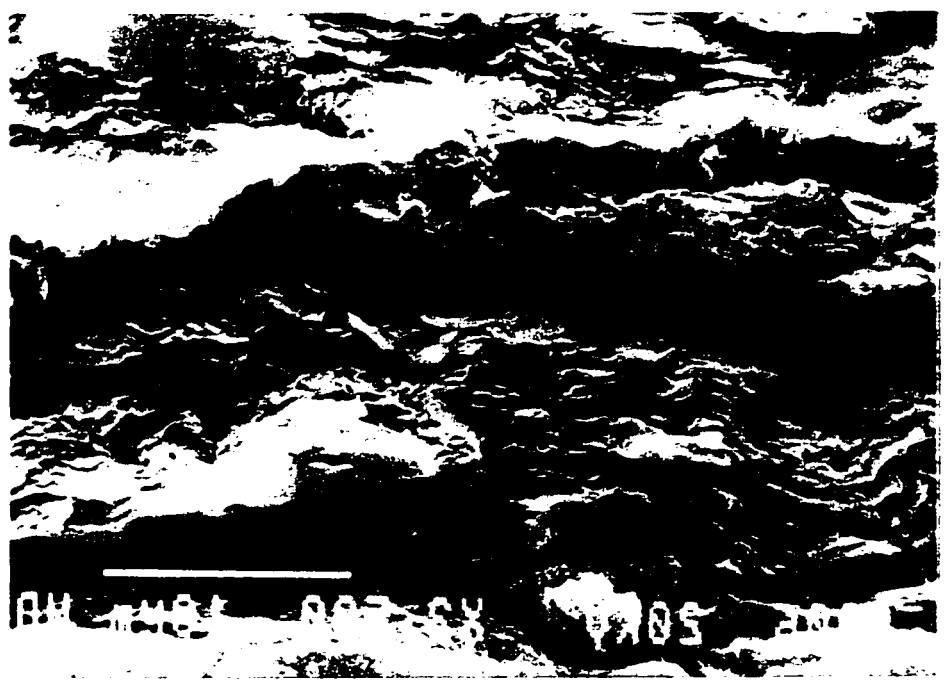


Fig. 4.11 : Scanning Electron Photomicrographs of Undisturbed Soil Fabrics for Samples from Block # 04. Upper photograph showing fabric along layers, lower one perpendicular to layers.

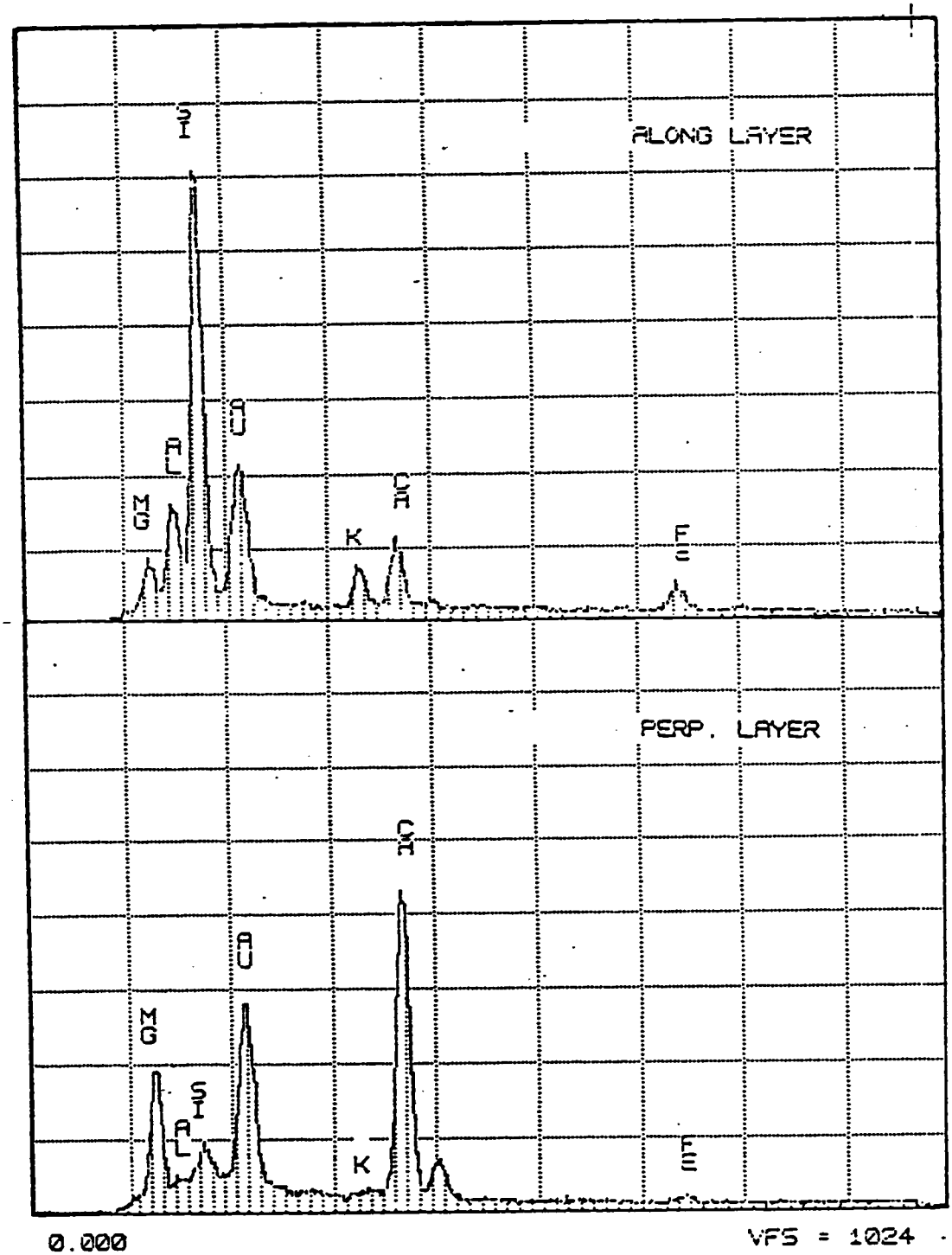


Fig. 4.12 : EDS for Soil Samples from Block # 01 Showing Peaks for Different Elements Present

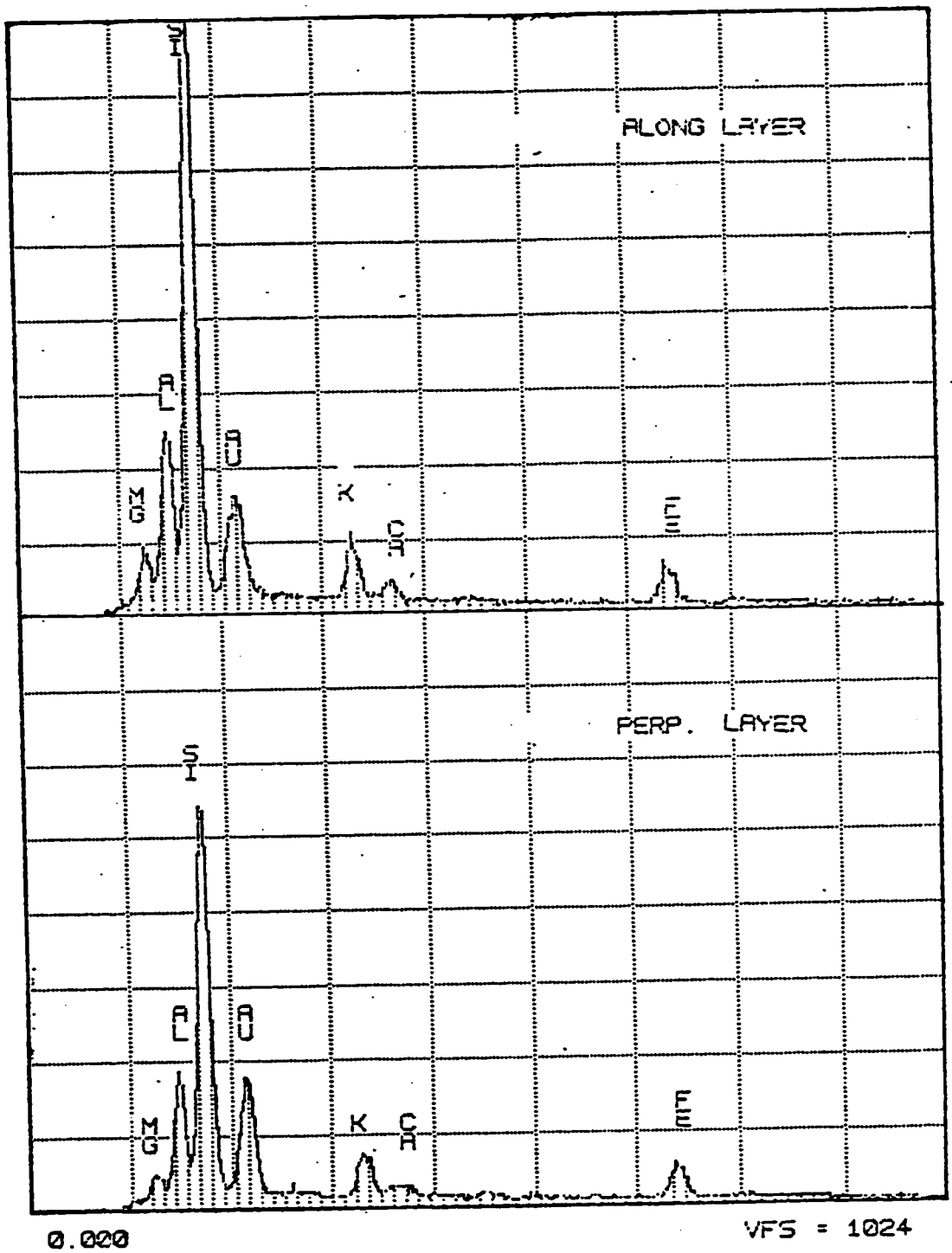


Fig. 4.13 : EDS for Soil Samples from Block # 02 Showing Peaks for Different Elements Present

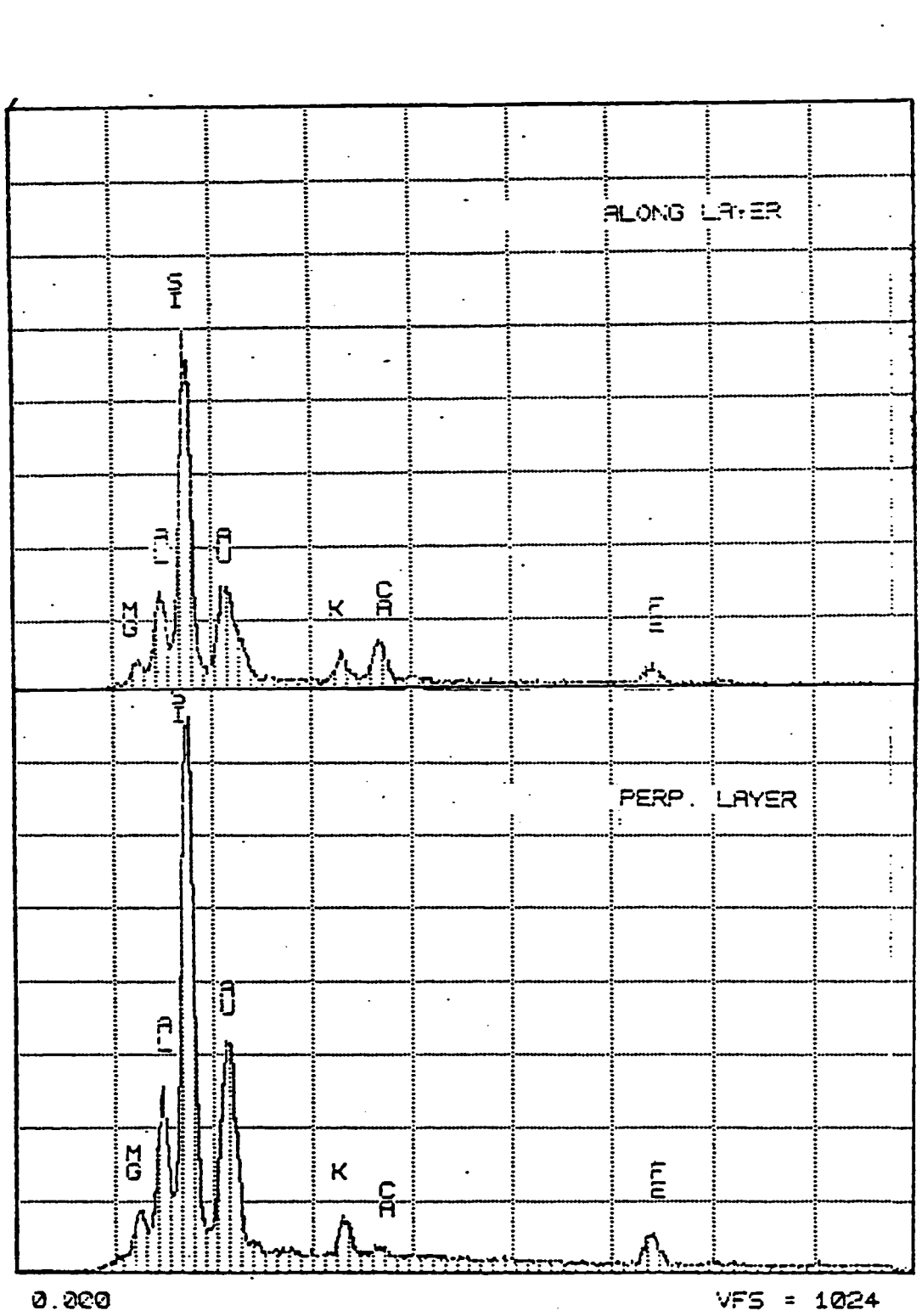


Fig. 4.14 : EDS for Soil Samples from Block # 03 Showing Peaks for Different Elements Present

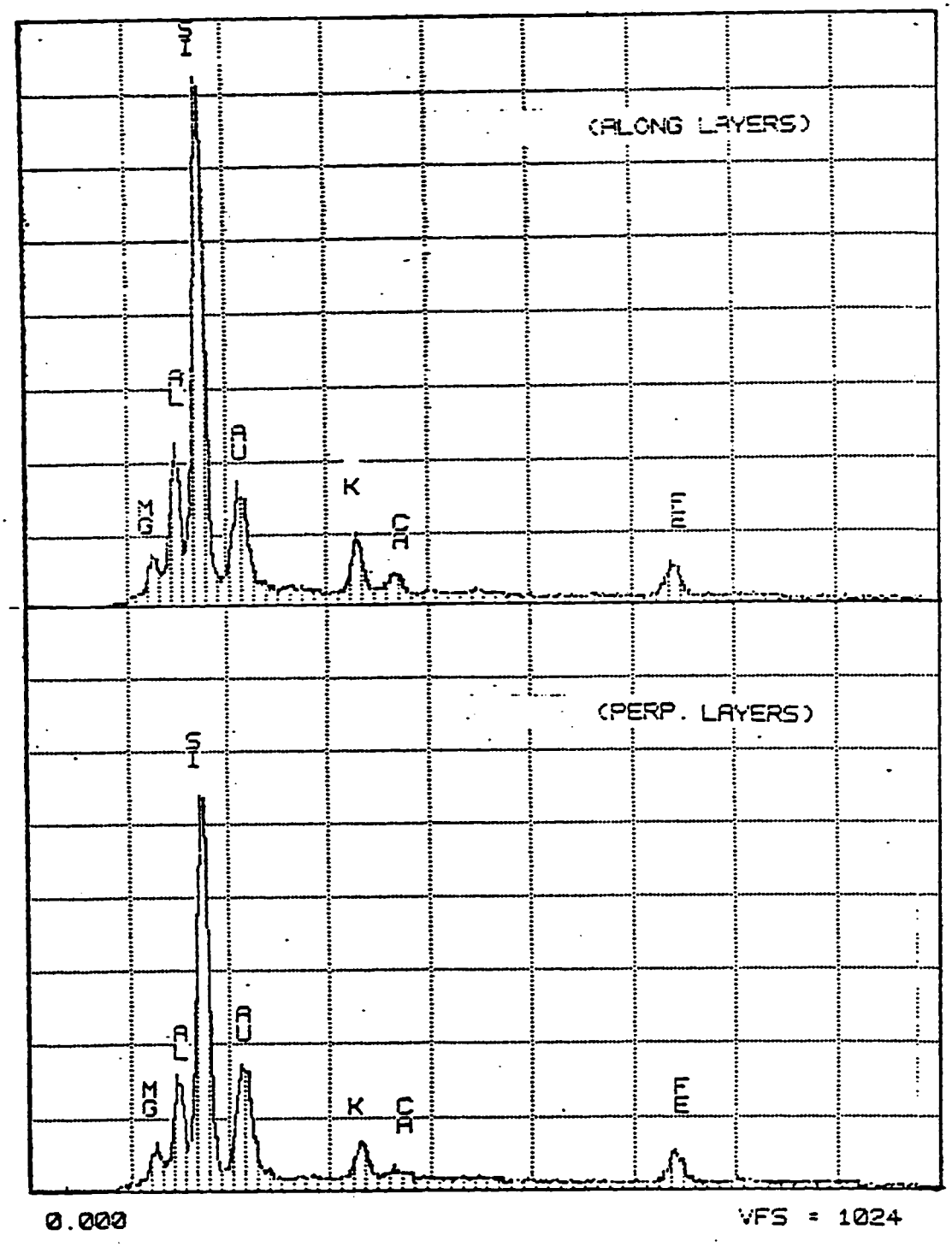


Fig. 4.15 : EDS for Soil Samples from Block # 04 Showing Peaks for Different Elements Present

The presence of polygorskite (fibers morphology) was confirmed by SEM photomicrographs perpendicular to layers for some samples. These needle shaped particles are sandwiched between the Na-montmorillonite flakes, Fig. 4.9(b) and 4.11(b). For sample from block # 02, these needle shaped particles can also be seen in photomicrograph parallel to layers, Fig. 4.9(a). Although polygorskite is present in samples from all the four soil blocks as indicated by x-ray diffraction analysis results, Table 4.1, but the fiber morphology is not clear in some of the photomicrographs merely because same magnification was maintained throughout SEM analyses. For a soil sample, a certain kind of morphology is possible at a particular magnification.

The Energy Dispersive Spectra (EDS) depicted in Figs. 4.12 to 4.15 indicate only the elements present in the corresponding soil sample and not the minerals. As shown in Table 4.1, dolomites (Ca-Mg rich particles) were present only in samples from block # 01. This fact is confirmed by EDS for the same sample from block # 01 as depicted in Fig. 4.12 where high intensity Ca & Mg peaks are reported. For soil samples from other three blocks EDS show very low peak intensity of Ca & Mg, predicting absence of dolomite. High percentages of Quartz is confirmed by high Si-peak intensity.

4.6 Determination of Organic Matter, CEC and pH

The term "soil organic matter" embraces the whole non-mineral fraction of soil and any vegetable or animal matter forming part of the sample. Organic matter contributes to the physical condition of a

soil by holding moisture and by affecting structure. The presence of large amounts of organic matter in soils is usually undesirable from an engineering standpoint. Organic matter may cause high plasticity, high shrinkage, high compressibility, low permeability, and low strength [9].

Organic matter in soils is complex both chemically and physically and varies with age and origin (Kononova, 1961; Schmidt, 1965). It may occur in any of the five groups: carbohydrates; proteins, fats; resins and waxes; hydrocarbons; and carbon. Cellulose ($C_6H_{10}O_5$) is the main organic constituent of soil. In residual soils organic matter is most abundant in surface horizons. Organic particles may range down to 0.1 μm in size. The specific properties of the colloidal particles vary greatly depending upon parent material, climate and stage of decomposition [12].

ASTM D-2974-34 procedure was followed to determine the total organic matter. Four samples from the four blocks were analyzed. The presence of organic matter was readily detected by treatment of soil with a 15% hydrogen peroxide solution. The H_2O_2 and organic matter react to give vigorous effervescence. Total organic carbon is reported as such and total organic matter is expressed by use of a multiplication factor, Table 4.2. The percentage of organic matter in any of the samples is not very high. The maximum fraction of total organic content (0.7629%) was reported in the sample from block # 03. No fossils or presence of any other organism was detected. The

Table 4.2 : Organic Matter Content & cec of Al-Qatif Soil Samples

Sample from	Cation exchange capacity cec (meq/100 g)	Organic Carbon (%)	Total Organic matter (%)	pH
Block # 01	24.1	0.2025	0.3491	7.91
Block # 02	23.6	0.0300	0.0517	7.90
Block # 03	24.0	0.4425	0.7629	7.91
Block # 04	22.5	0.3000	0.5172	7.89
Total organic matter = 1.724 x (organic carbon)%				

most probable source of this small amount of organic matter was found to be plant debris. This was confirmed by distinct color of the soil specimen after treating with H_2O_2. Plants might have dumped in the area during early stages of decomposition converting to colloidal material. Since Al-Qatif area is located on the sea shores of Arabian Gulf, the most probable source of this material may be the sea weeds. Although the percentage of organic matter is low but even these percentages have considerable influence on the properties of soils.

Following procedure was adopted for the determination of cation exchange capacity of soil samples [12].

Weigh 5 g of soil into a 50 cm^3 centrifuge tube, add 30 cm^3 of sodium acetate solution & shake for 5 minutes. The tubes should be stoppered with polythene or clean rubber stoppers and not corks which introduce errors. Centrifuge the tubes at 200 rev./sec. for about 5 minutes until the liquid is clear. Discard the liquid and repeat the shaking and centrifuging four times more with fresh portions of acetate solution. Shake the soil with 30 cm^3 of 95% ethanol for 5 minutes, centrifuge and discard the liquid. Repeat the ethanol washing three times. Finally extract the soil with three 30 cm^3 portions of ammonium acetate solution and collect the extracts in a 100 cm^3 graduated flask. Dilute the combined extracts to 100 cm^3 and determine the sodium content. Calculate the value of cec for each soil sample as:

$$\text{cec (meg/100 g of soil)} = \frac{10 \times \text{Na concentration}}{\text{weight of sample}}$$

Table 4.2 also shows the values of cation exchange capacities of the four soil samples studied. If we compare these cec values with those of clay minerals, Table 2.1, we find that they fall in the range of pure illite. This means that most soil samples possess high cec values contributing to high expansion on wetting. Actually cation exchange capacity (cec) is an imperfect indication of montmorillonite content. This might be due to the presence of high percentages of Na-montmorillonite as reported in Table 4.1. Expandability and hence activity of montmorillonite is greatly influenced by the size and charge of the interlayer cation. Thus sodium or hydrogen montmorillonite is highly expandable; calcium and magnesium montmorillonite exhibit controlled expansion [28].

The logarithmic of the reciprocal of the hydrogen ion concentration in a solution is termed as pH. The determination of the acidity or alkalinity of a soil sample in terms of pH is a relatively simple measurement carried out using a pH-meter or special indicator (Peech, 1965; ASTM D-2976, 1981). A soil suspension is acidic if its pH is less than 7, and basic or alkaline if pH is more than 7. A solution is neutral if its pH is exactly 7. Low exchange capacities are measured under low pH conditions and high exchange capacities are obtained at high pH [12].

The electrometric measurement of the pH of soil samples from

the four blocks was made in suspensions of water and calcium chloride solutions. The measurement was made using a pH-meter with a glass-calomel electrode system, calibrated with buffers of known pH. The pH values for the four soil samples has been reported in Table 4.2. The results show that all soil samples were alkaline in nature as the pH values range between 7.89 to 7.91. These high pH values support high cec values for the same soil samples.

CHAPTER 5

SWELLING CHARACTERISTICS

5.1 Introduction

Quantitative estimation of expansion in clays can be carried out using direct swelling tests. Three different types of swelling tests were performed in this study. Those are:

(1) Swell Pressure Tests

(2) Swell Percentage Tests, and

(3) Consolidation Tests on Initially Unsoaked Samples.

5.2 Swell Pressure Tests

In this study swell pressure tests were performed on samples taken from the four collected soil blocks. The blocks were unwrapped in a humid room to prevent loss of natural moisture content. Paraffin wax was removed by a sharp edged knife as shown in Fig. 5.1. After confirming the layering system, the consolidometer ring was pushed down carefully to get an undisturbed sample as shown in Fig. 5.2. Since most of the soil blocks were quite wet due to high natural moisture content, it was easy to get undisturbed samples just by pushing the ring with hands. Extra soil was removed from the top of the ring to make the sample height equal to the height of the ring.

Fig. 5.1 : Identification of Layering System of the Blocks

Fig. 5.2 : Sample Preparation for Swell Pressure Tests

The sample enclosed in 6.9 cm x 1.9 cm brass ring was placed in the odeometer between two porous stones. Water was introduced into the odeometer and the sample was flooded both from the top and bottom. The sample was restrained against swelling and the maximum load recorded by load cell connected with the data logger divided by the cross-sectional area of the specimen was regarded as the swell pressure in (kg/cm^2). The load cell was connected to a portable data logger (TDS-301) for accurate measurements and automatic recording of the pressure exerted by the specimen on the load cell. A line diagram for swell pressure test setup is shown in Fig. 5.3.

Figs. 5.4 and 5.5 show the swell pressure versus time for samples taken from the four different blocks collected from Al-Qatif. These two diagrams very well document the expansivity range of these clays. Fig. 5.4 simply shows the maximum values of swell pressure while the same data drawn on Fig. 5.5 using a semi-logarithmic plot depicts the process of swelling with time more clearly. As shown in Fig. 5.5, the whole phenomenon can be divided into three distinct stages: initial, intermediate and final which represent the instantaneous, primary and secondary swelling. In the initial stage when water just enters the pores, not much expansion was observed. When water gets way into the pores, progressive swelling starts and continues till soil's ability to maximum volume change. The maximum swelling pressure of 3.25 kg/cm^2 was exerted by soil sample from block # 03. This high value of swell pressure is attributed to low natural water content & high plastic limit values as shown

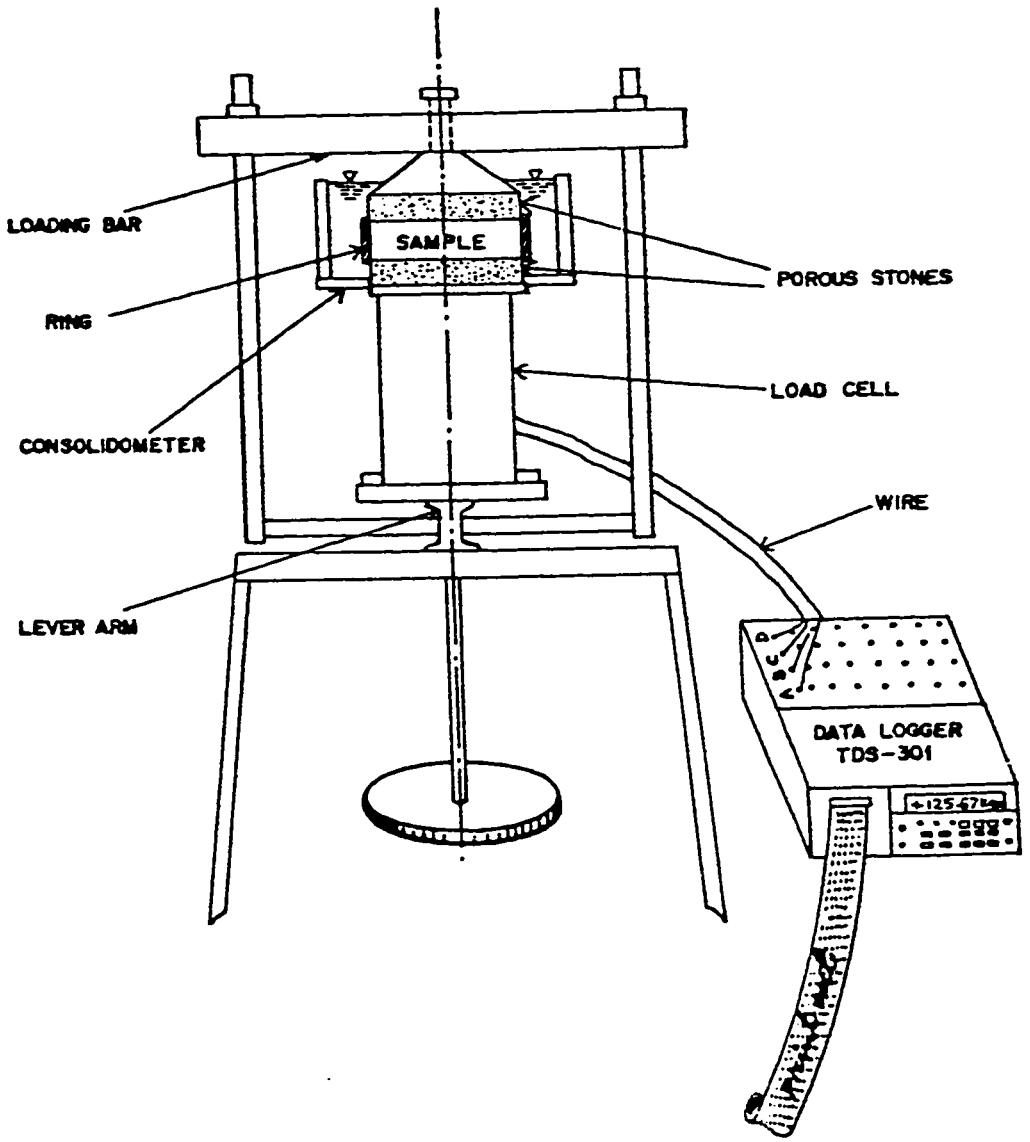


Fig. 5.3 : Swell Pressure Test Apparatus

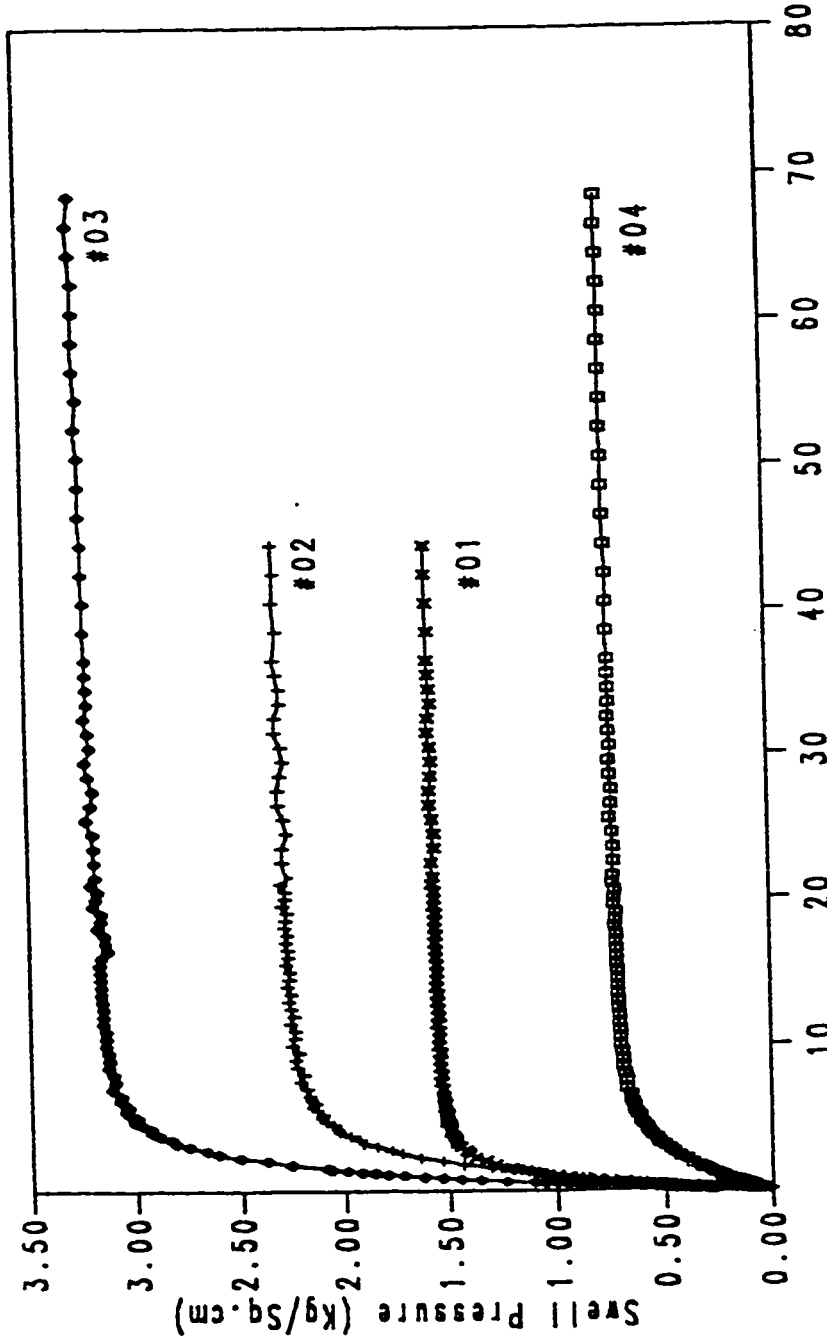


Fig. 5.4 : Swell Pressure vs Time Curves for Samples from Al-Qatif Area (K.S.A.) (Arithmetic Scale)

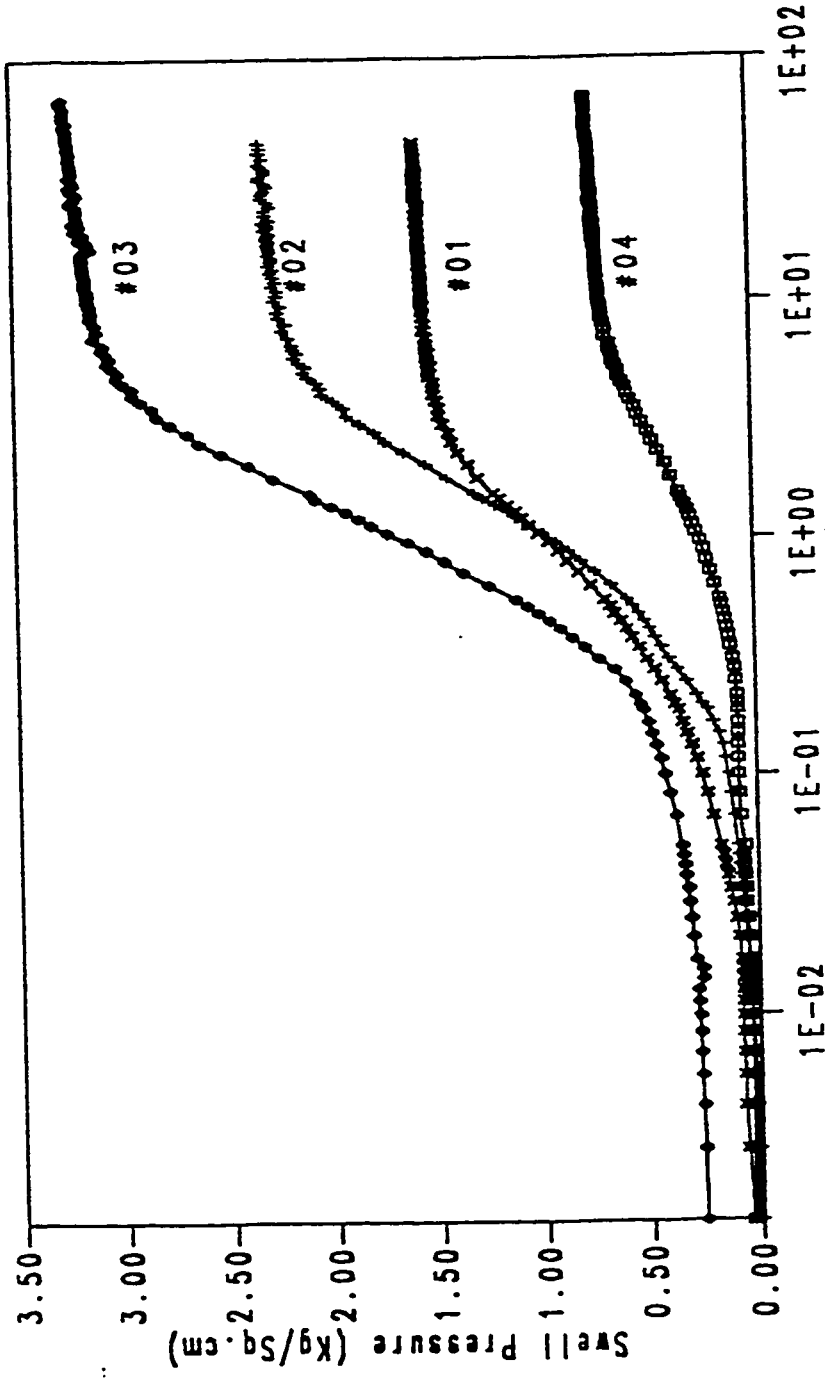


Fig. 5.5 : Swell Pressure vs Time Curves for Samples from Al-Qatif (K.S.A.) (Semi-log Plot)

in Tables 3.1 and 3.2 respectively. For sample from block # 02, the swelling pressure is sufficiently higher because of its high silt & clay contents as shown in Fig. 3.7. Sample from block # 01 is possessing a low water content and theoretically this sample should exhibit high swelling pressure. But from Table 4.1, it is clear that block # 01 contains low percentage of montmorillonite and flocculant arrangement of particles, thus sample from this block is showing low swelling pressure. Sample from block # 04 shows a low swelling pressure because the natural water content is greater than the plastic limit and the percentage of clay is low.

The curves shown in Figs. 5.4 and 5.5 demonstrate the relative degree of expansiveness of the four samples studied, but do not necessarily reflect the magnitudes of in-situ heave which is extensively influenced by site conditions such as climate, drainage system and type of foundation, etc.

5.3 Effect of Various Physical Properties on Swell Pressure

Swelling tests were performed by altering some physical properties to see their effect on swell pressure. Results show that block # 03 can be characterized as the most expansive out of the four blocks. Part of this block was broken into small pieces. Then these broken pieces were compacted to natural density by static loading device called "versa tester". Results of swell pressure tests performed on both remolded and undisturbed specimens are shown in Figs. 5.6 and 5.7. The final swell pressure is higher for

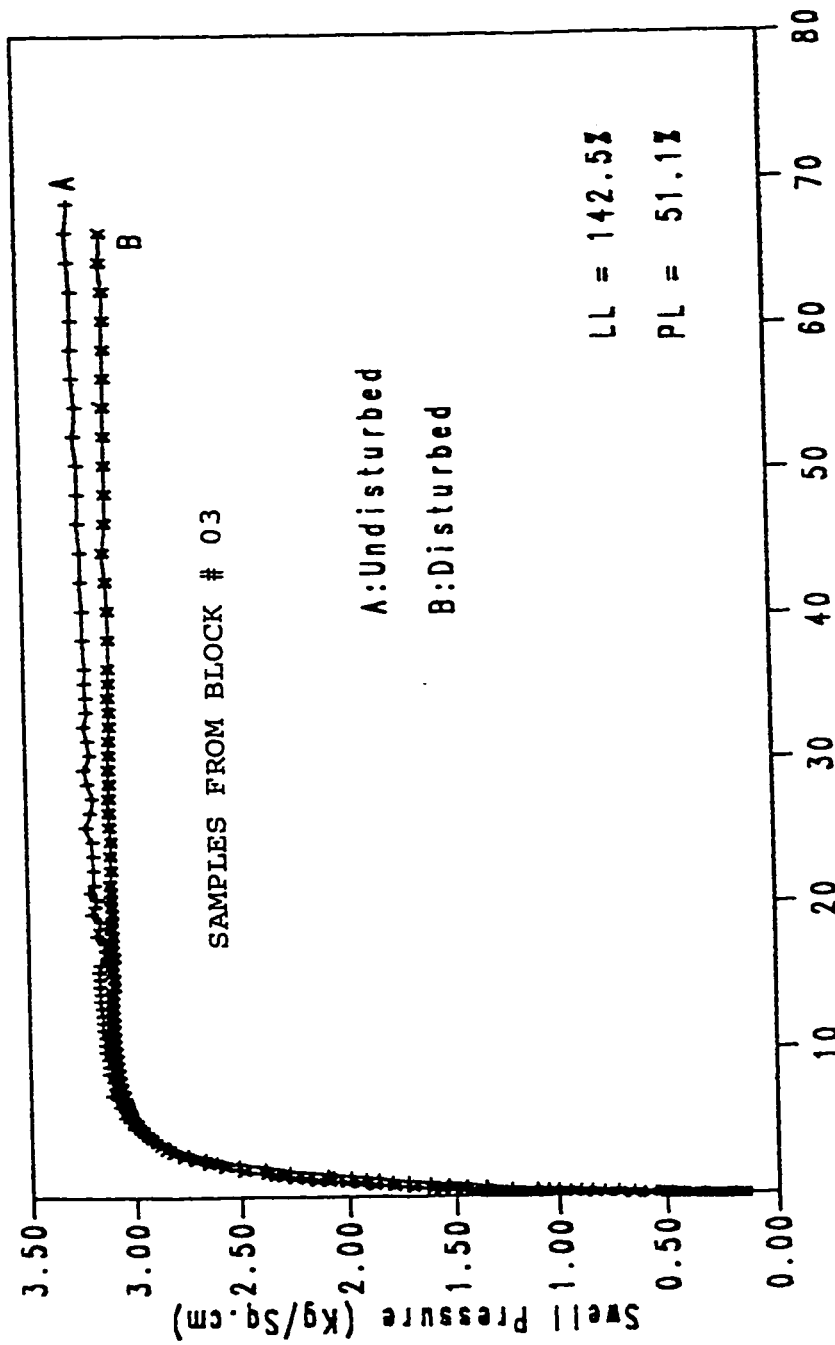


Fig. 5.6 : Swell Pressure of an Undisturbed Sample as Compared with that of a Remolded Specimen (Arithmetic Scale)

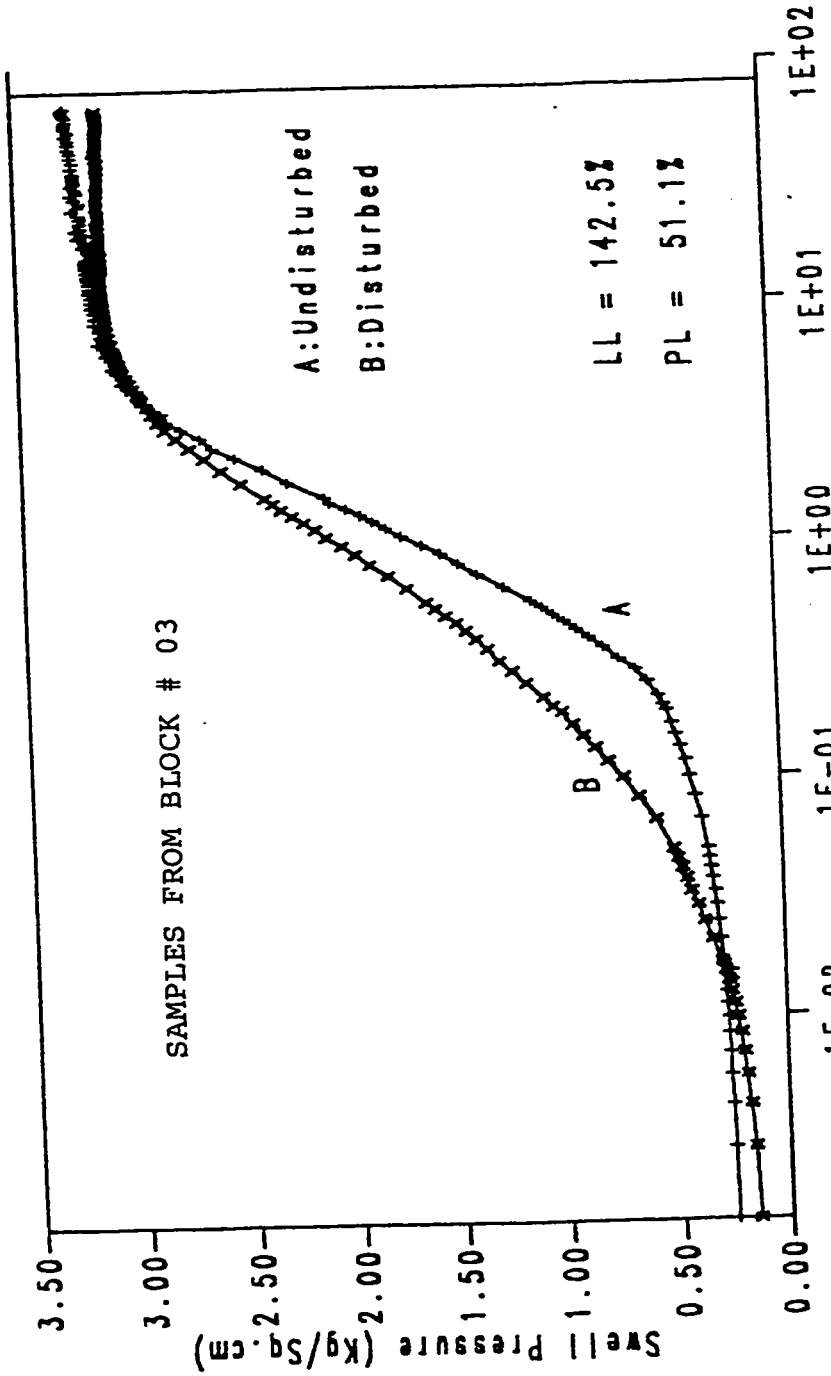
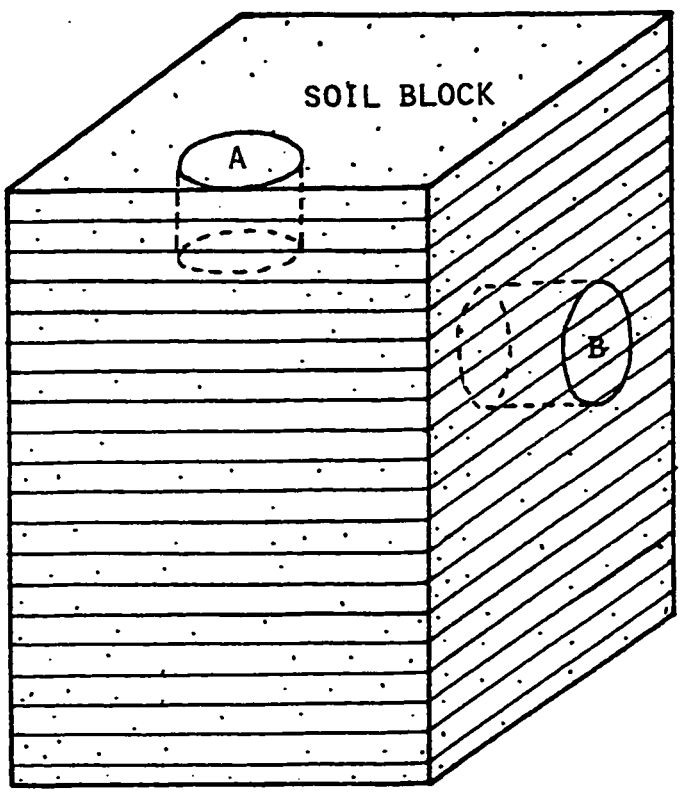


Fig. 5.7 : Swell Pressure of an Undisturbed Sample as Compared with that of a Remolded Specimen (Semi-log Plot)

undisturbed sample. In case of remolded sample, the soil fabric was more flocculated and the sample was unable to swell to its peak value. In the intermediate stage as depicted in Fig. 5.7, the swell pressure shown by the remolded specimen is more than that of undisturbed one because the disturbance will alter the geometry and the structural arrangement of clay particles causing large size air voids. Finally the undisturbed soil sample having more oriented grains exerts more pressure on the load cell as compared to the disturbed one.

A soil sample swells more in the direction normal to platlets than parallel to platlets. When water enters the pores in a soil sample with horizontal layering as shown in Fig. 5.8, clay layers expand, exerting sufficient pressure on the load cell. But if the layering system is altered and a sample with vertical layering is tested, a low swell pressure value will be attained because the sample will try to expand in the directions normal to the platlets but is restrained by the sides of the solid ring. This fact was confirmed by the results shown in Figs. 5.9 and 5.10 for samples from block # 03 & # 04 respectively.

Samples from block # 03 were tested in dense, natural and loose states. The dense sample was prepared by placing more amount of soil than that in the natural state and compacting to the same volume by static compaction. Similarly loose sample was prepared by using lesser amount of soil than natural state and compacting in the



A : Soil sample with Horizontal Layering

B : Soil sample with Vertical Layering

Fig. 5.8 : Soil Samples with Horizontal & Vertical Layerings taken from a Soil Block

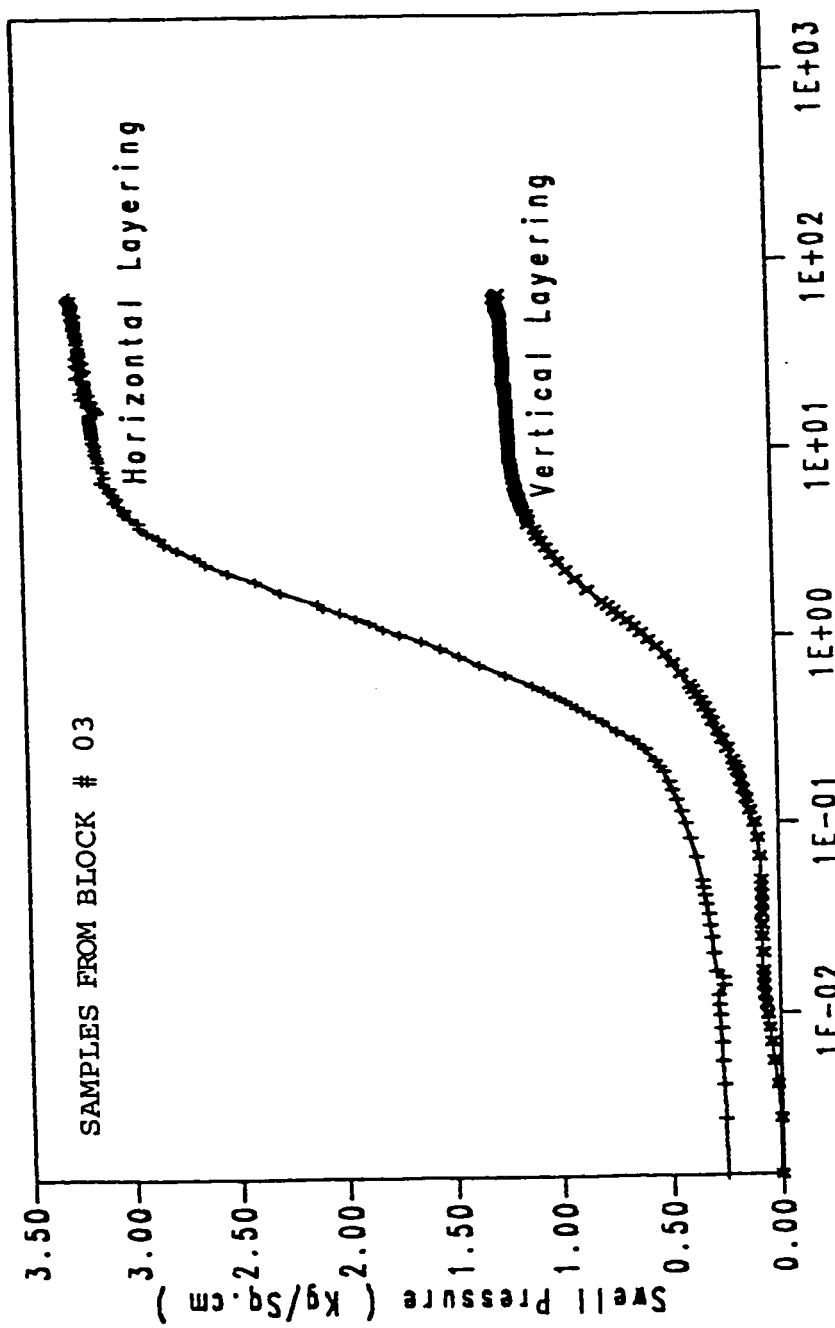


Fig. 5.9 : Comparison of Swell Pressure for Horizontal and Vertical Layering (Block # 03)

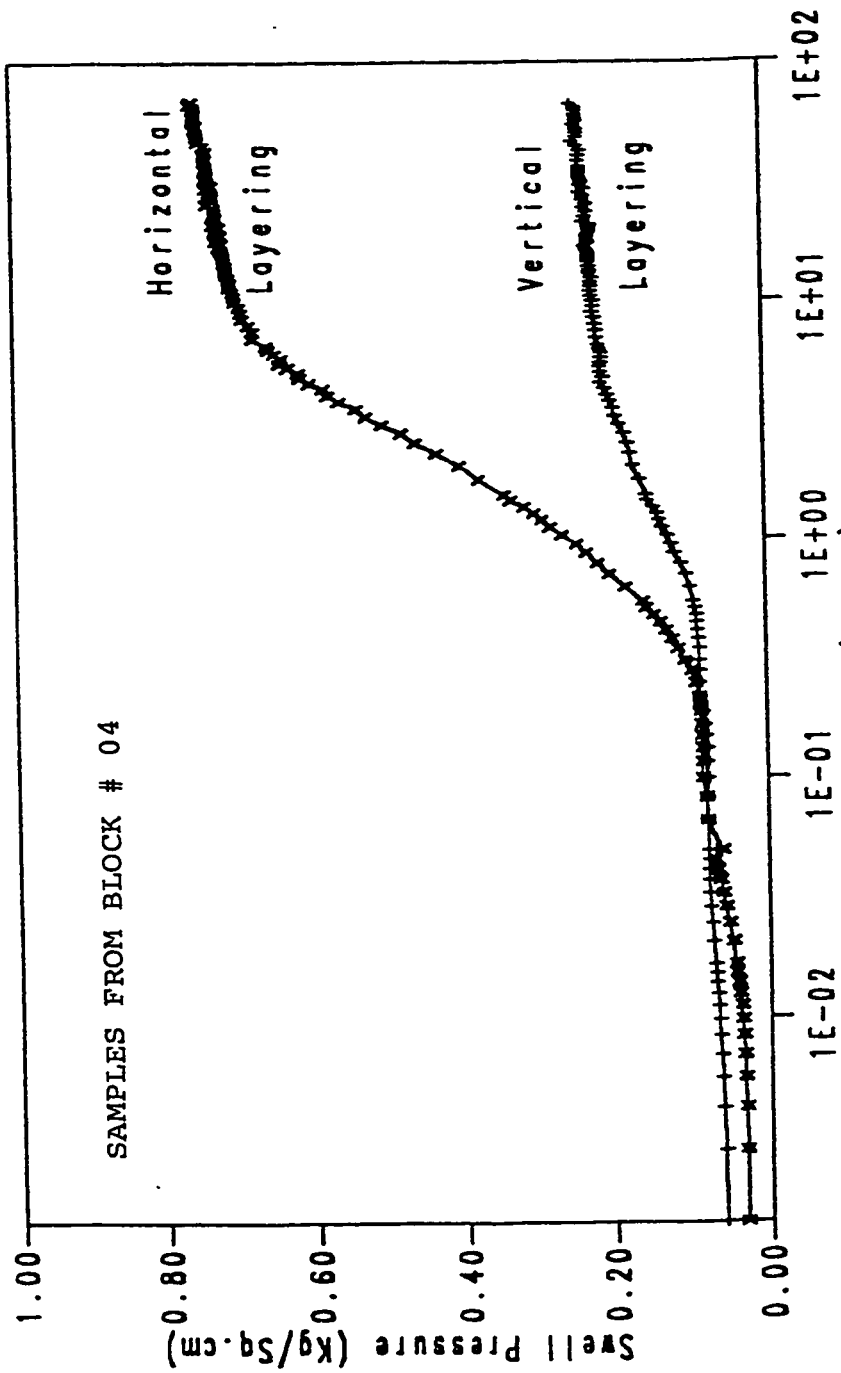


Fig. 5.10 : Comparison of Swell Pressure for Horizontal and Vertical Layering (Block # 04)

same way. The actual bulk density of the sample was 1.66 g/cm^3 while the density of the dense & loose samples were 1.81 and 1.53 g/cm^3 respectively. The results of the three tests are plotted in Figs. 5.11 and 5.12. From these curves two conclusions may be drawn that the sample in the dense state possesses higher swell pressure than in loose state, while the actual state lies in between. Secondly, as indicated by Fig. 5.12, a looser soil sample may possess high swell pressure in the initial stage because of the larger void sizes.

Effect of moisture content was checked by using two soil samples from the same block, at the same dry density, one on the dry side of the optimum while other on the wet side of the optimum as shown in Fig. 5.13. The idea was to keep the dry density same for specimens below and above the optimum moisture content. Different parameters were calculated and summarized in Table 5.1. The results of the two tests as shown in Figs. 5.14 and 5.15 agree with the actual practice in the field. Most materials engineers and consultants propose to compact expansive soils in the field preferably above optimum moisture content. This technique helps in proper compaction and also reduces the expansion. In the initial stage sample above optimum swells more than the sample which was prepared below optimum because the former one contains more water in the voids. Table 5.2 lists the results of the effect of different physical properties on swelling pressure.

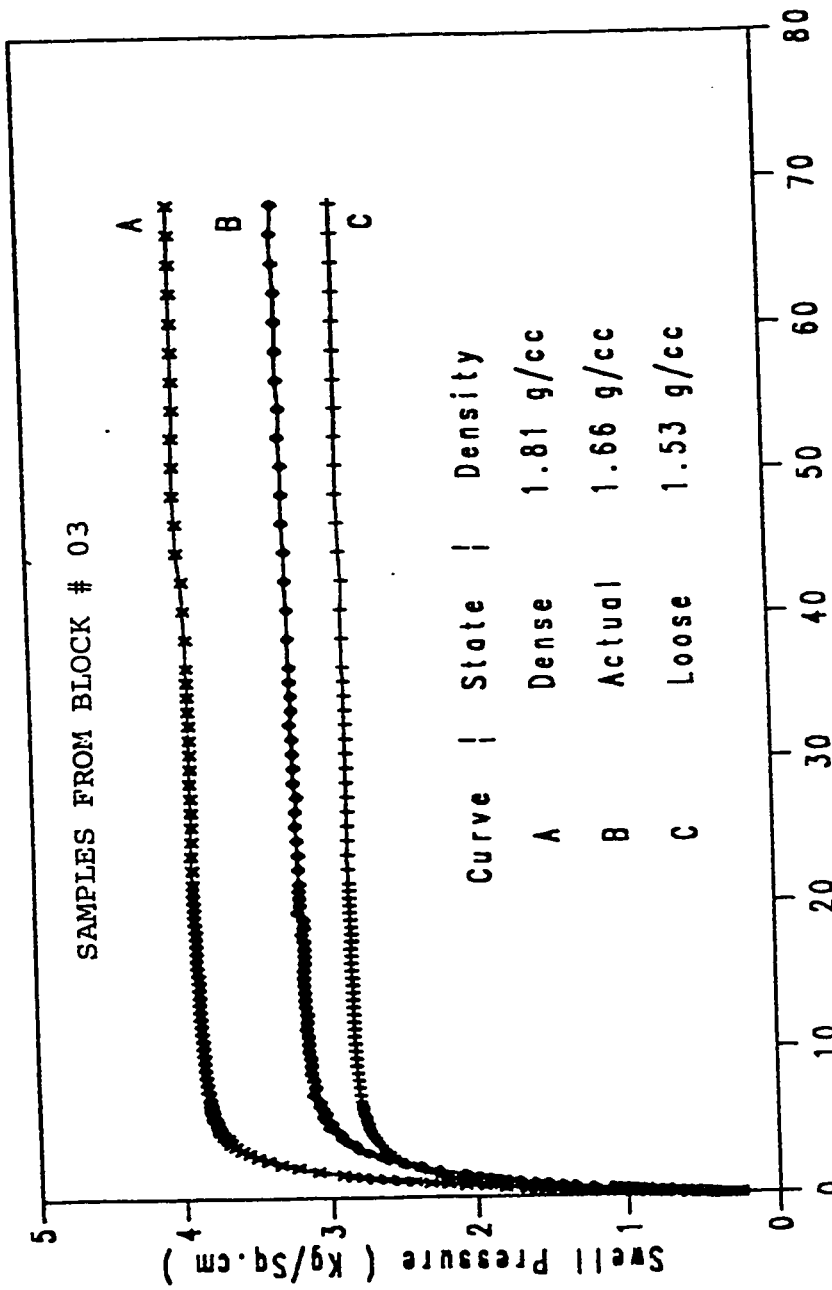


Fig. 5.11 : Effect of Density of Swelling Pressure (Arithmetic Scale)

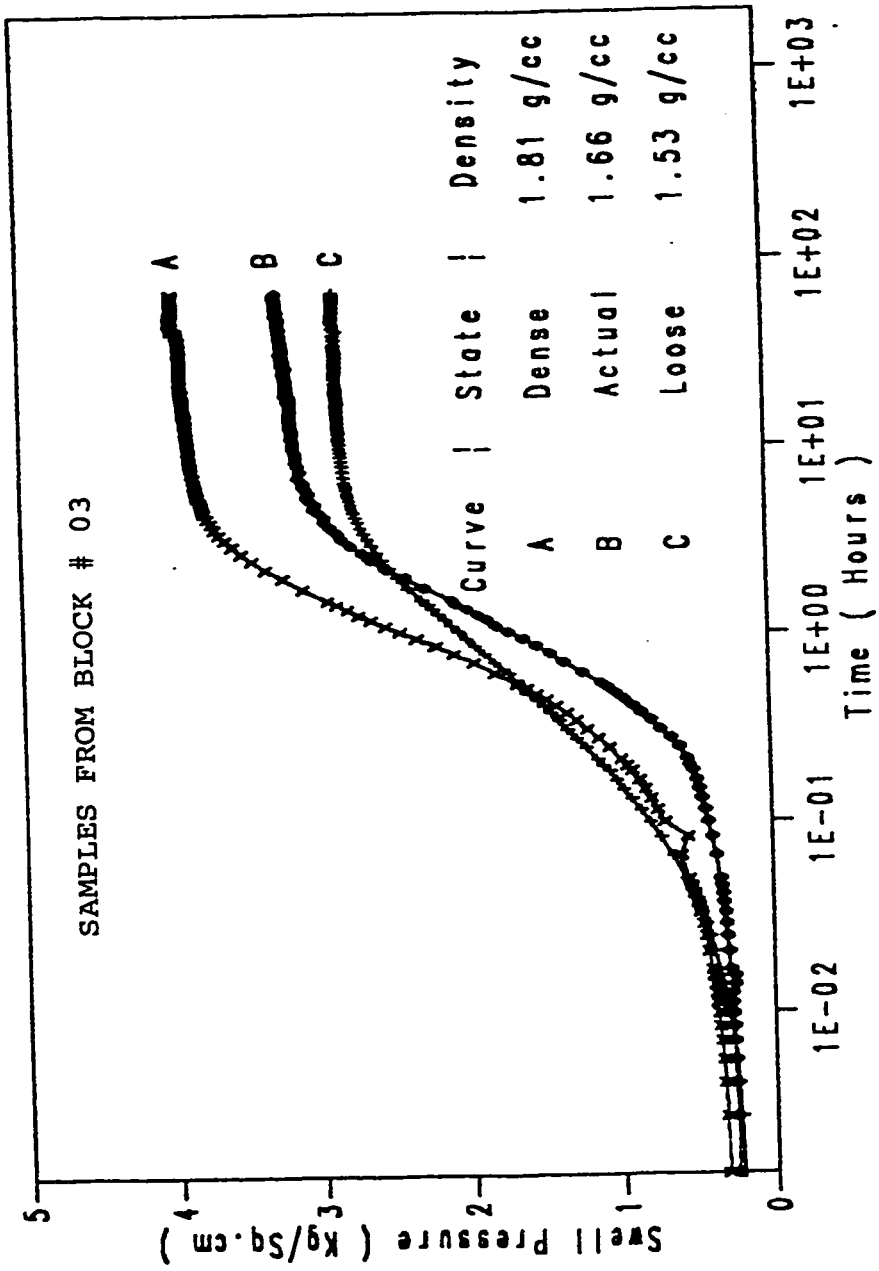


Fig. 5.12 : Effect of Density on Swelling Pressure (Semi-log Plot)

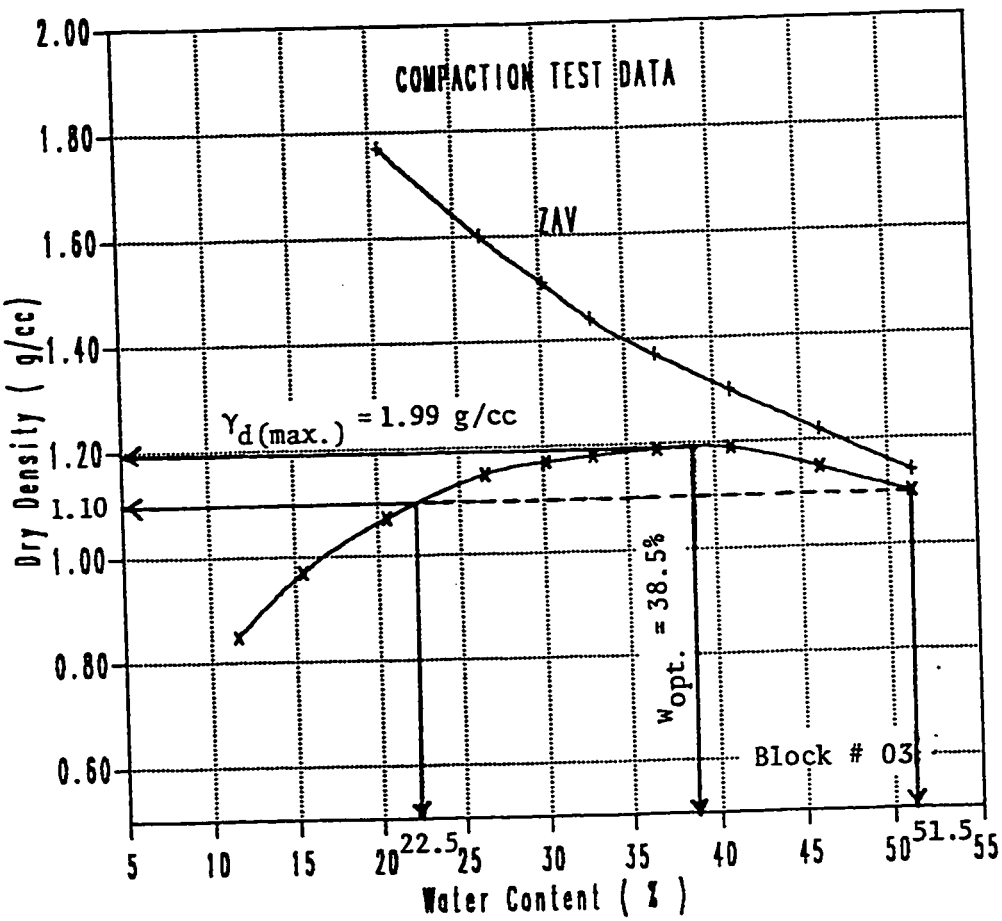


Fig. 5.13 : Use of Compaction Curve to Test the Sample from Block # 03 below and above Optimum Moisture Content

Table 5.1 : Calculations for Swell Pressure Test for Sample from Block # 03 Below & Above Optimum Moisture Content

Stage	(w) w.c.	w.c. already existing in the sample	(w*) w.c. needed to get req'd. moisture	Dry density γ_d	Wet density $\gamma_{wet} = \gamma_d(1+w)$	Weight of Sample $W = V \times \gamma_{wet}$	$w = \frac{W_w}{W_s} \times 100$ Water needed $w^* = \frac{W_w}{W - W_w} \times 100$	Swell Pressure (Kg/cm^2)
(a)	(b)	(c)	(d) = (b) - (c)	(e)	(f)	(g)	(h)	(i)
Below $w_{opt.}$	22.5%	9.7%	12.8%	1.1 g/cc	1.35 g/cc	98.60 g	9.8 mℓ	3.81
Above $w_{opt.}$	51.5%	9.7%	41.8%	1.1 g/cc	1.66 g/cc	121.49 g	35.8 mℓ	2.26

Internal volume of the ring = Vol. of sample (V) = 73.08 cm^3

Optimum moisture content w_{opt} = 38.5% (from compaction curve)

Maximum dry density $\gamma_{d(max)}$ = 1.99 g/cc (from compaction curve)

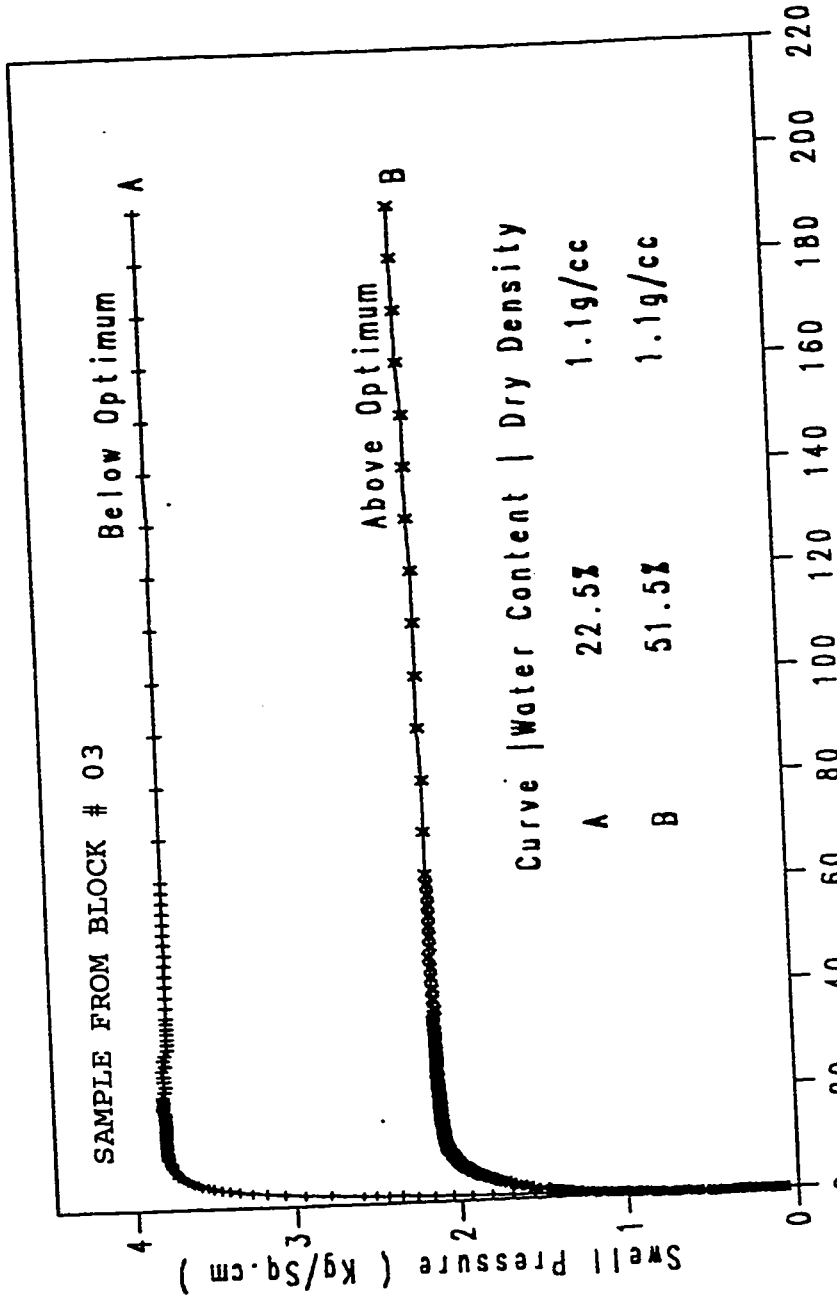


Fig. 5.14 : Effect of Moisture Content and Dry Density on Swelling Pressure (Arithmetic Scale)

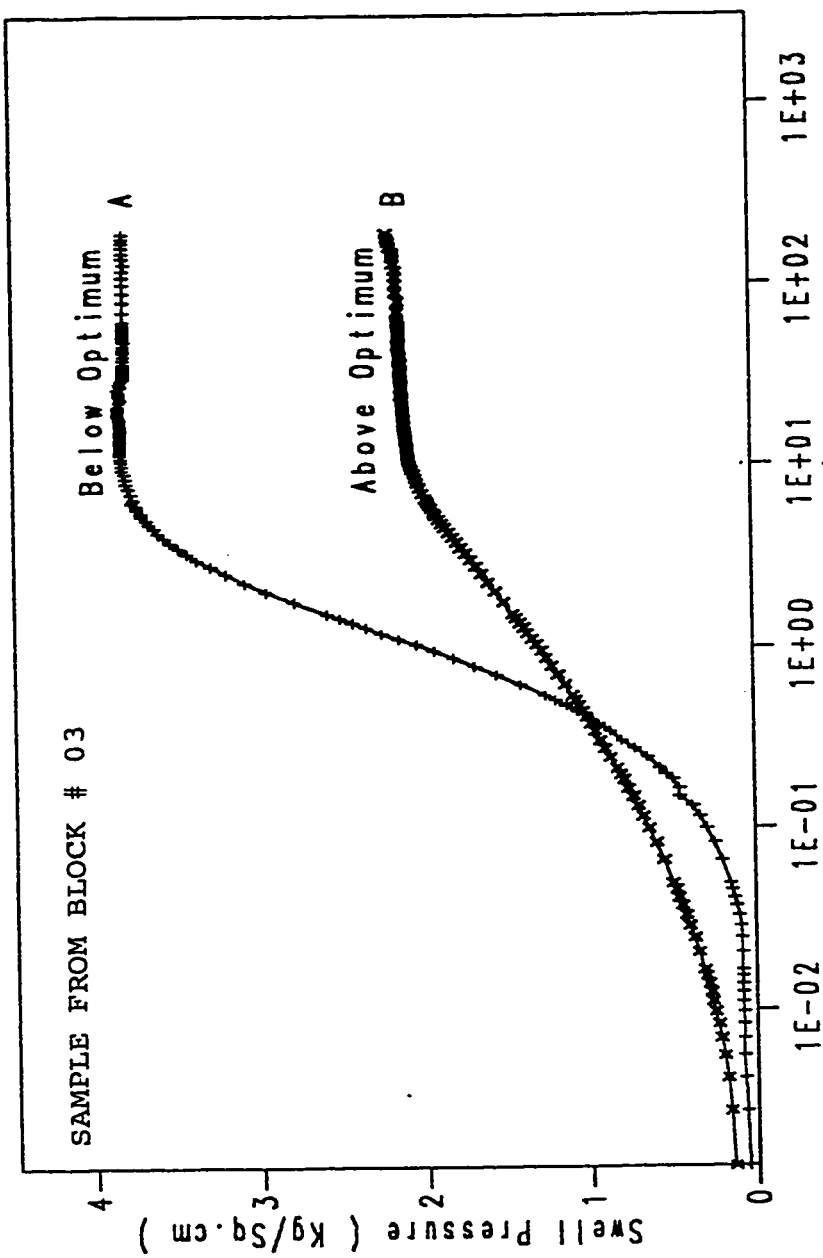


Fig. 5.15 : Effect of Moisture Content and Dry Density on Swell Pressure (Semi-log Plot)

Table 5.2 : Comparison of Swell Pressure Values for Various States of Soil Samples

Sample from	State	Swelling Pressure (Kg/cm^2)	Reference
Block # 03	undisturbed	3.25	Figs. (5.4) & (5.5)
Block # 02	undisturbed	2.31	
Block # 01	undisturbed	1.58	
Block # 04	undisturbed	0.73	
Block # 03	undisturbed disturbed	3.25 3.01	Figs. (5.6) & (5.7)
Block # 03	horiz. layering vert. layering	3.30 1.29	Fig. (5.9)
Block # 04	horiz. layering vert. layering	0.73 0.26	Fig. (5.10)
Block # 03	dense actual loose	4.01 3.25 2.83	Fig. (5.11) & (5.12)
Block # 03	below $w_{opt.}$ above $w_{opt.}$	3.81 2.26	Figs. (5.14) & (5.15)

5.4 Swell Percentage Tests

Swell percentage tests are also known as swell potential or unrestrained swell tests. This type of tests essentially involve direct measurement of the vertical rise of a soil sample placed under some surcharge and in contact with water inside a consolidometer.

Various investigators used different surcharge pressures. A surcharge pressure of 6.9 KPa (1 psi) has been used for swell tests by Holtz and Gibbs (1954), Seed, Woodward and Lundgren (1962), Kassif and Holland (1965), while Chen (1975) indicated that the swell is very sensitive to changes in pressure and the use of low surcharge pressure (less than 1 psi) may lead to erratic and erroneous results. Chen therefore recommended a surcharge load of about 50 KPa (7.2 psi) simulating the pressure that most footing foundations are likely to exert on the soil. The results of swelling tests are also known to be influenced by other factors, including: initial moisture content and dry density, size of soil specimens, and method of compaction (Chen, 1975; Seed, Mitchell and Chen, 1962).

Fig. 5.16 shows the percent swell versus time for undisturbed soil specimens taken from the four different blocks collected from Al-Qatif. A surcharge pressure of 1 psi was maintained throughout the test. All specimen were kept unrestrained during testing. Volume changes readings were recorded by the dial gauge attached to LVDT. Swell percentage was calculated as the percentage of vertical rise of soil specimen to original sample height (1.9 cm).

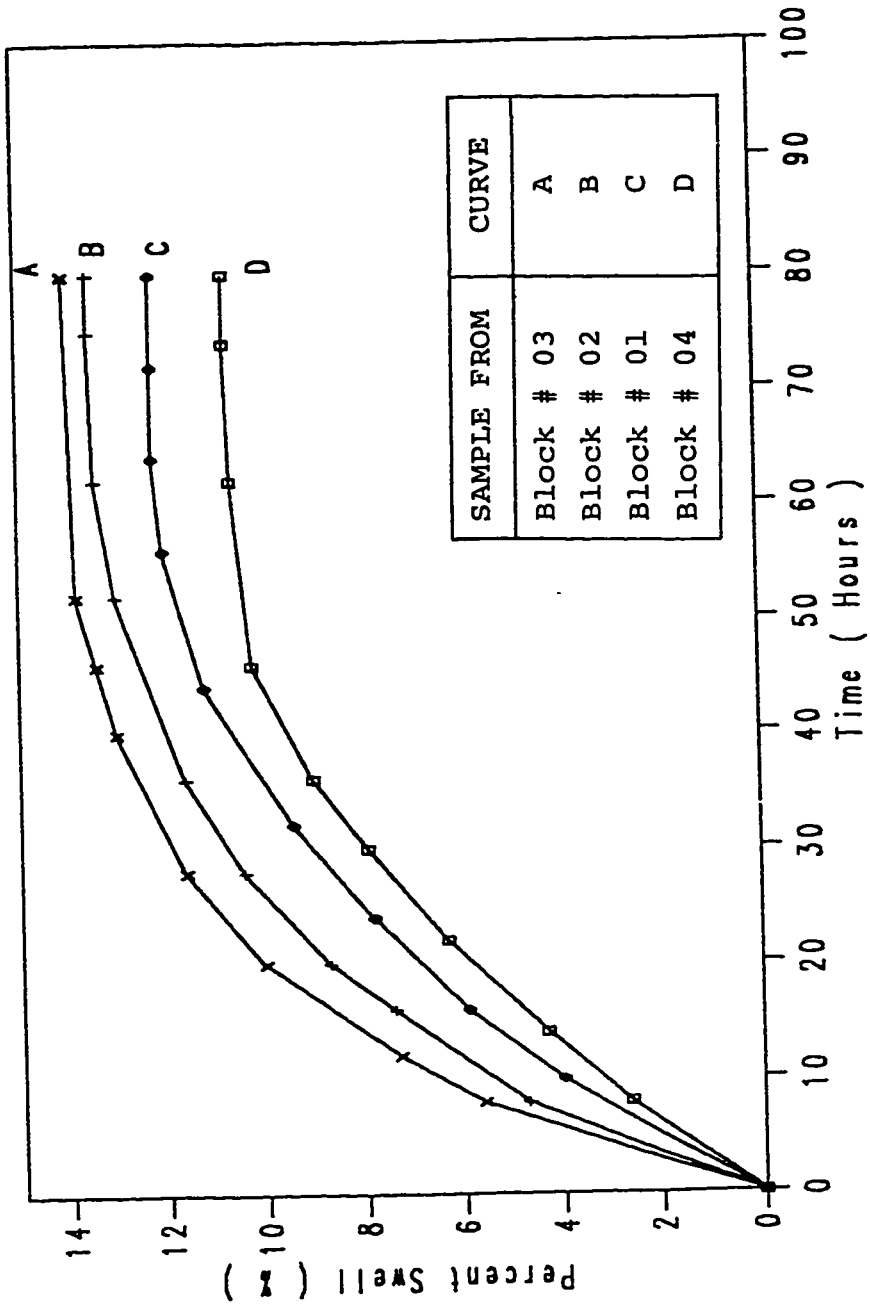


Fig. 5.16 : Percent Swell versus Time Curves for Al-Qatif Soil Samples

After reaching to their maximum volume change under 1 psi surcharge, all samples were loaded in small increments. The loading was continued till the samples attained their original height of 1.9 cm or zero percent swell, Fig. 5.16. Swell pressure was calculated as follows [2]:

$$\text{Swell Pressure} = \frac{\sum P}{A}$$

Where,

$\sum P$ = load increments ($P_1 + P_2 + P_3 + \ldots$)

A = the cross-sectional area of the specimen (38.5 cm^2)

As shown in Fig. 5.16, the values of swell pressure measured in these swell percentage tests are somewhat smaller than those obtained for corresponding samples in swell pressure tests in section 5.1. The reason behind this fact is that the specimens in swell percentage tests are kept unrestrained while in swell pressure tests the samples are tested in restrained conditions exerting more pressure on load cell.

As clearly seen in Figs. 5.16 and 5.17, the swell potential of Al-Qatif clays are characterized as high, occasionally within medium to high range. This is in agreement with the measured swell pressure values in section 5.2. The magnitude of percent swell reaches to more than 14% and swell pressure is as high as 3.13 Kg/cm^2 for undisturbed soil specimens from block # 03, as shown in Figs. 5.16

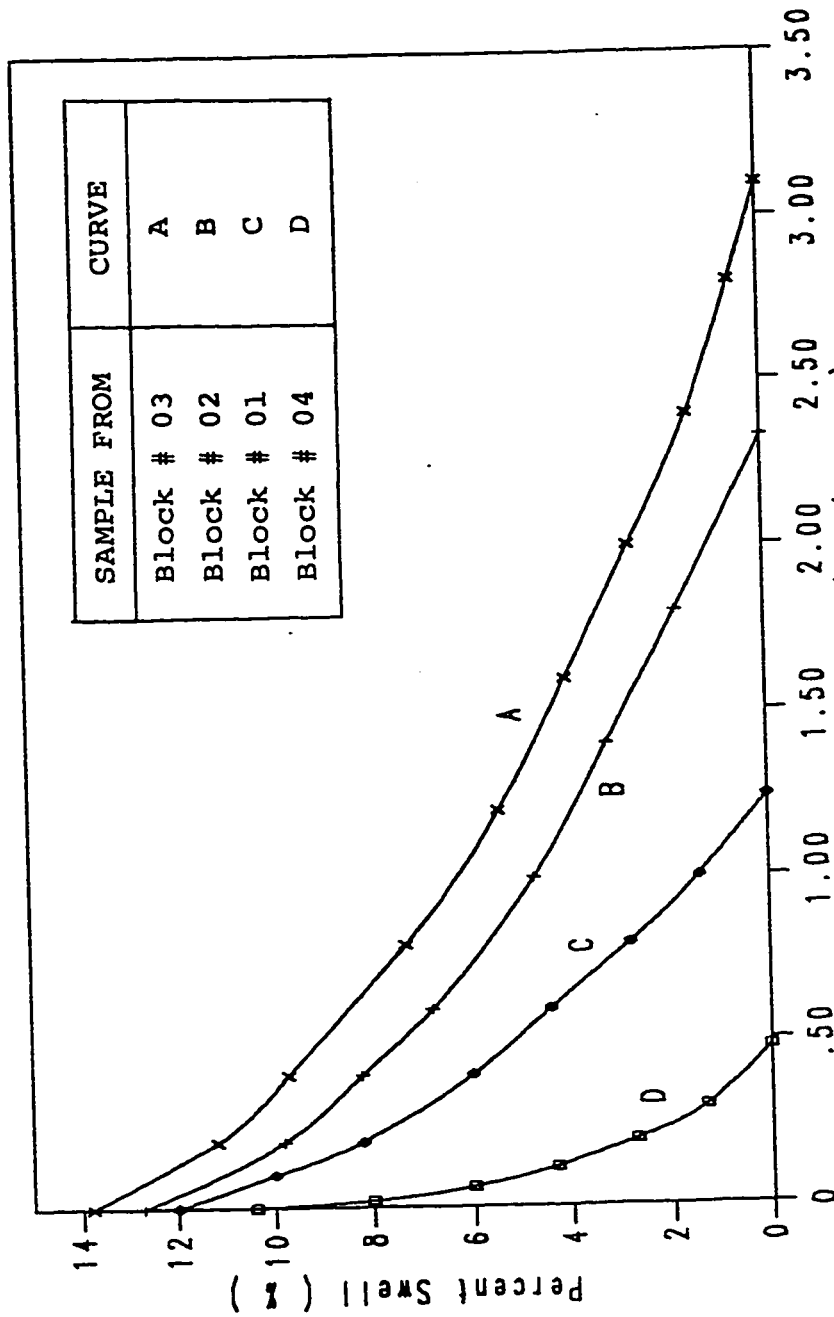


Fig. 5.17 : Relationship between Percent Swell and Swell Pressure for Soil Samples Studied

and 5.17. The swell pressure value in this case is somewhat smaller than that obtained in swell pressure test on soil sample from the same block # 03. The overall behavior of samples is very similar to that obtained earlier in swell pressure tests. The only difference is that little lesser values of swell pressure are obtained in swell percentage tests due to surcharge pressure & unrestrained conditions.

5.5 Consolidation Tests with Initially Unsoaked Samples

Samples from block # 03 & # 04 were tested unsoaked. In this case, the sample with natural water content was subjected to an initial small surcharge pressure of 2.5 psi. After some time consolidometer was filled with water and the sample was allowed to soak. It was observed that the deformation dial gauge readings shot up as soon as the soil sample got in touch with water. During this process no further load was applied until full saturation or in other words until full swelling. When there was no change in dial gauge readings, the sample was compressed by prescribed load increments followed by unloading in steps. The test on one sample took about 14 days to complete. Void ratio versus the logarithmic of effective pressure curves are depicted in Figs. 5.18 and 5.19. The percentage swell defined as the ratio of the increase in height of the sample to its initial height prior to soaking was 10.9% for sample from block # 03 and only 7.5% for sample from block # 04.

From e-log p plots, it is possible to calculate the %swell and swell pressure. Swell pressure is obtained by drawing a line parallel

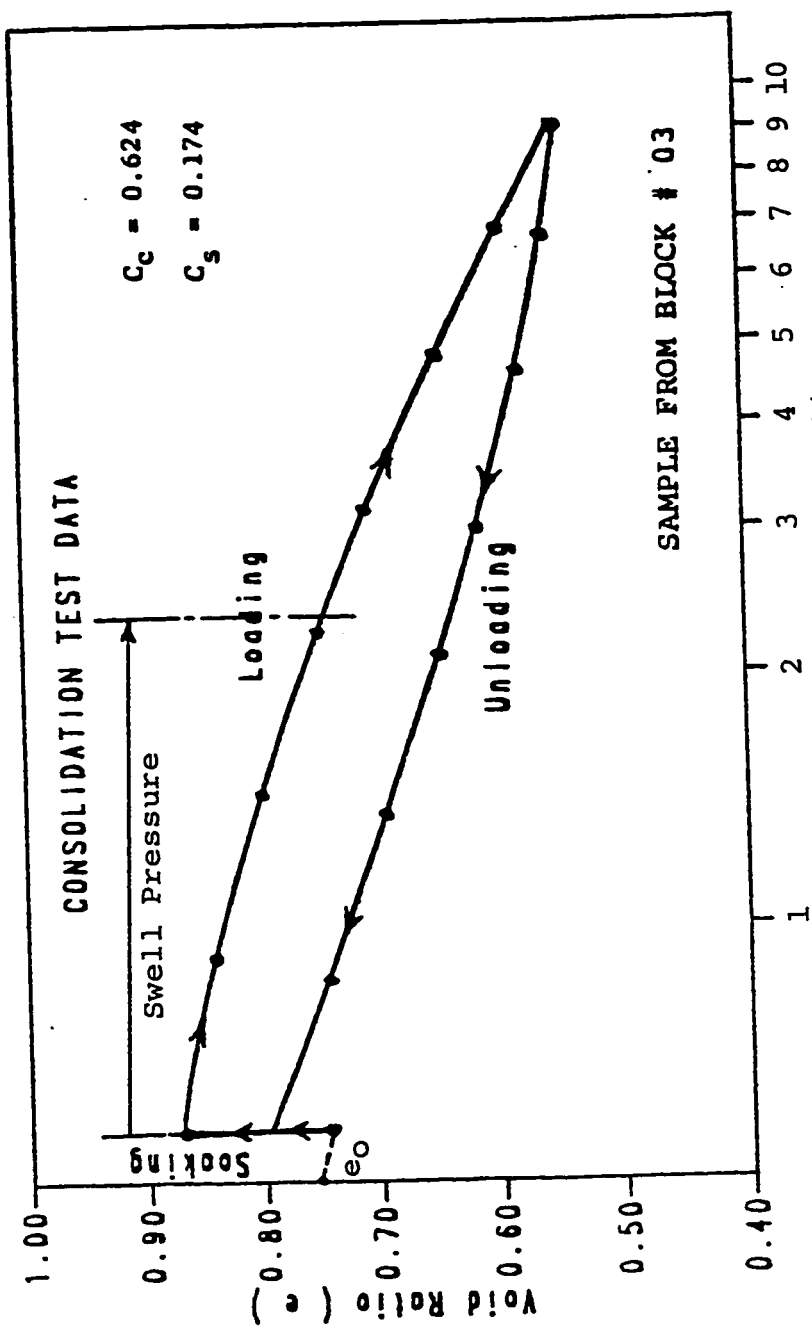


Fig. 5.18 : Void Ratio-log Pressure Relationship for Initially Unsoaked Sample from Block # 03

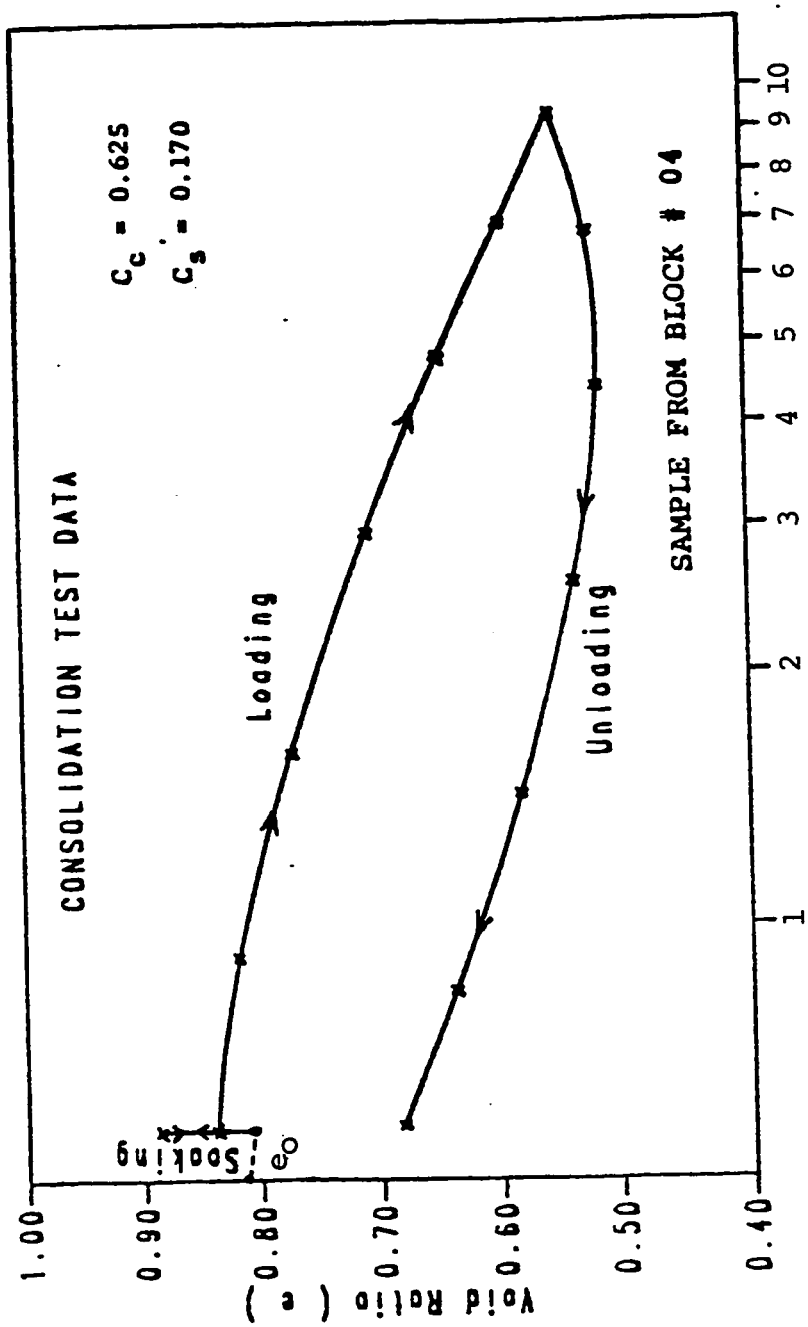


Fig. 5.19 : Void Ratio-log Pressure Relationship for Initially Unsoaked Sample from Block # 04

to x-axis from the soaking point & extending until it intersect the loading curve. The projection of the horizontal distance between the soaking point and the intersection on the x-axis gives the swell pressure as shown in Fig. 5.18.

Table 5.3 summarizes the results of the three types of swelling tests performed on Al-Qatif soil samples. A comparison can be made between swell pressure values and %swell values. Several conclusions can be drawn with the help of Table 5.3. This table verifies the fact that there is a loss of natural moisture content with time even when the soil blocks are completely covered. The three tests were conducted in sequence as shown in Table 5.3. A slight decrease in natural moisture content was observed. This might be due to some time lag between taking samples from the blocks and rewaxing them. The swell pressure values obtained in swell percentage tests are somewhat smaller than those in the swell pressure tests. As regards %swell values, these are higher in case of swell percentage tests as compared to consolidation tests performed on samples from the same blocks. This happened due to comparatively smaller surcharge (1 psi) in case of swell percentage tests, allowing the specimen to swell more. Comparing the results shown in Table 2.5 with the results obtained by various investigators, will indicate that Al-Qatif clays have medium to high swelling potential.

Table 5.3 : Summary of Laboratory Swelling Test Results on Al-Qatif Soil Samples

Swell Pressure Tests					
Sample from	w.c. (%)	γ_{wet} (g/cm^3)	% Swell	Swell Pressure (Kg/cm^2)	Reference
Block # 01	30.2	1.84	-	1.55	Figs. 5.4 & 5.5
Block # 02	51.9	1.79	-	2.31	
Block # 03	52.0	1.90	-	3.25	
Block # 04	57.9	1.95	-	0.61	
Swell Percentage Tests (1 psi surcharge)					
Sample from	w.c. (%)	γ_{wet} (g/cm^3)	% Swell	Swell Pressure (Kg/cm^2)	Reference
Block # 01	29.5	1.83	12.1	1.25	Figs. 5.16 & 5.17
Block # 02	50.8	1.77	13.6	2.28	
Block # 03	51.9	1.87	14.0	3.13	
Block # 04	56.7	1.93	10.8	0.49	
Consolidation Tests on Initially Unsoaked Samples (2.5 psi surcharge)					
Sample from	w.c. (%)	γ_{wet} (g/cm^3)	% Swell	Swell Pressure (Kg/cm^2)	Reference
Block # 03	50.0	1.91	10.9	2.15	Figs. 5.18 & 5.19
Block # 04	52.1	1.94	7.5	0.43	

5.6 Comparison Between Experimental Results and Actual Swelling Behavior and Movement in the Field

Following effects should be taken into account while extrapolating laboratory results to the behavior of expansive clays in the field.

Effect of Confinement

The laboratory swelling test is one-dimensional because the samples are confined in a metal ring with no soil movement or water flow in the lateral directions. In the field some radial water flow may occur, as well as some slight lateral soil movement. So, the laboratory test may overestimate the swelling pressure and percentage of swell due to complete confinement.

Effect of Thickness

In the laboratory a small portion of soil mass is used to simulate the field conditions. It is not necessary that this small sample truly represents the soil mass under study because even in a very small area the soil properties may differ. Soil strata may possess fissures & hard rocks at places. Also, swelling of soil is directly proportional to the thickness of the expansive soil. The thickness of lab. sample is usually quite small compared to the actual thickness of the expansive soil layers in the field.

CHAPTER 6

CHEMICAL STABILIZATION

6.1 Introduction

The term soil stabilization is applied to any process which improves the properties of a soil and which enables it to perform and sustain its engineering use (Winterkorn, 1955). The strength, bearing capacity and durability of soils can be increased by adding inorganic and organic compounds. These chemicals act chiefly as cementatious binding agents and also help in reducing plasticity of expansive clays [9].

Inorganic compounds used include portland cement, pure lime, commercial lime, phosphorous compounds and sometimes combinations of these. Various inorganic salts such as chlorides of calcium, sodium and potassium have long been used for stabilization (Johnson, 1958; Slate, 1961; Bowles, 1979). Most of these chemicals are effective under laboratory conditions, but their application in the field is very difficult because of their high cost. There is no supporting evidence that any of these chemicals is economically feasible except commercial lime & portland cement [30].

In the present study, pure lime, commercial lime, potassium chloride and potassium phosphate have been used for laboratory evaluation of chemical stabilization of expansive soils collected from

Al-Qatif area.

6.2 Lime Stabilization

Lime is one of the oldest stabilizing agents known to man. Lime stabilization was used as a roadway stabilizer by the Romans as well as other early civilizations. In the United States, lime stabilization was first used in Texas in 1940 for clayey base materials. Now lime is the most widely used stabilizer throughout U.S.A. and other developed countries [15].

In general, the term lime refers to oxides and hydroxides of calcium and magnesium, but not to carbonates. There are various types of limes commercially available. Calcitic quicklime (CaO) and dolomitic quicklime (CaO + MgO) are produced by heating calcitic and dolomitic limestones respectively. By controlled addition of water to quicklime, three types of hydrated lime can be produced: high calcium, $Ca(OH)_2$ monohydrated dolomitic, $Ca(OH)_2$ + MgO and dihydrated dolomitic $CA(OH)_2$ + $Mg(OH)_2$. Out of these, the most commonly used form is calcitic quicklime [30].

6.2.1 Soil-Lime Reactions

The addition of lime to a fine-grained soil initiates several reactions. Four important reactions are: (1) cation exchange; (2) flocculation/agglomeration; (3) carbonation; and (4) pozzolanic reactions. The first two increase soil workability and result from

changes in ionic charges of the clay minerals. The second two are cementation reactions, increasing bearing strength of the soil mass treated. In soil stabilization, the most important reaction is soil-lime pozzolanic reaction [30].

For a given compactive effort, soil-lime mixtures have a lower maximum dry density and a higher optimum moisture content than the untreated soil. Maximum dry density reductions of 3-5 pcf and optimum water content increases of 2-4 percent are common with 5-8% lime. Fig. 6.1 illustrates the effect of lime on the compaction characteristics of an expansive clayey soil (AASHTO T-99 compaction).

In addition to increasing bearing strength, lime decreases the plastic index of soils and greatly reduces the shrink-swell characteristics of highly plastic clays. It also increases the permeability by giving the clay silt-like mechanical properties and reeduces the maximum dry density.

6.2.2 Soil-Lime Pozzolanic Reactions

The reactions between lime, water, and various sources of soil silica and alumina to form cementing-type materials are referred to as soil-lime pozzolanic reactions. Possible sources of silica and alumina in typical fine-grained soils include clay minerals, quartz, feldspars, micas, and other similar silicate or alumino-silicate minerals [14].

A simplified geochemical mechanism for soil-lime pozzolanic reactions can be summarized by the following series of stepped

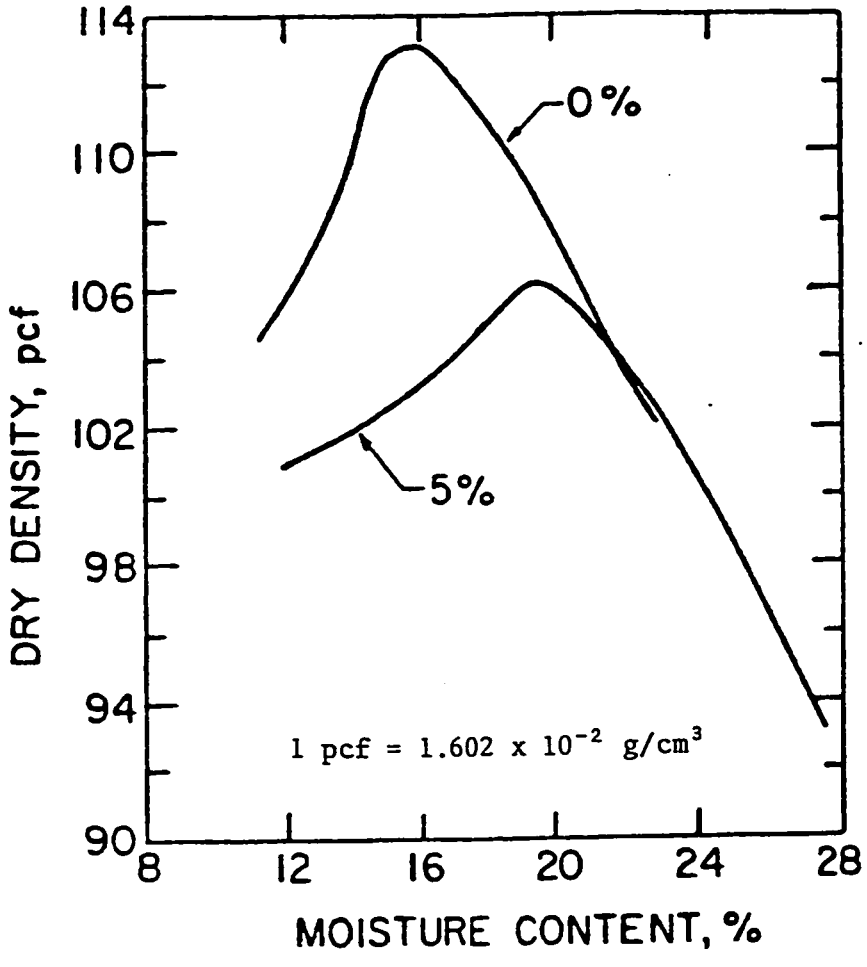


Fig. 6.1 : Moisture-Density Relations for a Natural and 5% Lime-Treated Expansive Soil (AASHTO T99 Compaction)

equilibrium reactions:

$$CaO + H_2O = Ca(OH)_2 \quad \text{(hydration of quicklime)} \tag{6.1}$$

$$Ca(OH)_2 = Ca^{+2} + 2(OH)^- \tag{6.2}$$

(ionization of calcium hyroxide; pH rises to 12.3)

$$Al_2Si_4O_{10}(OH)_2.nH_2O + 2(OH)^- + 10H_2O = 2Al(OH)_4 + 4H_4SiO_4 + nH_2O \tag{6.3}$$

(dissolution of clay mineral, at pH > 10.5)

$$2H_4SiO_4 = 2H_3SiO_4^- + 2H_2SiO_4^- + 2H^+ \tag{6.4}$$

(disassociation of silicic acid).

Eqs. (6.1) to (6.4) are normal pozzolanic reactions resulting in siliceous cementation of lime-treated soils. Eq. (6.3) is shown for montmorillonite, similar equation can be established for other clay minerals like illite and kaolinite, etc.

When a sufficient quantity of lime (2-8%) of its dry weight is added to soil, the pH of the soil-lime mixture is elevated to approximately 12.3, the pH of a saturated lime water as shown in eq. (6.2). This is a substantial pH increase compared to the pH of natural soils. The solubilities of silica and alumina are greatly increased at elevated pH levels, eq. (6.3). But when the reaction is complete, the pH drops and maintains a value around 10 [14].

Certain soil properties and characteristics influence the lime-reactivity of a soil. These include soil pH, organic carbon content, natural drainage, presence of excessive quantities of exchangeable sodium, clay mineralogy, degree of weathering, presence of

carbonates, extractable iron and silica-alumina products. If a soil is non-reactive, extensive pozzolanic strength development will not be achieved regardless of lime type, lime percentage or curing time [30].

6.2.3 Composition of Commercial Lime Used

In this study, a locally manufactured commercial lime was used for laboratory stabilization of Al-Qatif clays. Percentages of various constituents of commercial lime as shown in Table 6.1, were determined by the following methods:

Determination of pure lime fraction

1. Dry a portion of the lime sample at 103°C for 1 hr, cool in a dessicator and transfer to a dry weighing bottle.
2. Weigh the bottle and contents. Pour about 0.3 gms of the lime into a beaker and again weigh the bottle and contents. The difference in weight is the weight of the dry lime sample used.
3. Add about 30 ml of 50% HCl and heat gently for 5 minutes.
4. Filter and rinse the beaker and paper with small portions of hot distilled water.
5. Add 10 ml ammonium chloride (10%) solution to the filtrate and make it alkaline with NH_4OH.
6. If a precipitate of Fe and aluminium oxides forms, filter and wash the beaker and paper with distilled water.
7. Heat to boiling and add slowly with stirring 20 ml of a saturated solution of ammonium oxalate.

Table 6.1 : Composition of Commercial Lime Used

No.	Constituents	Percentage
1.	Percentage Calcium hydroxide $Ca(OH)_2$	66.23%
2.	" Magnesium oxide (MgO)	10.60%
3.	" Sodium	15.00%
4.	" Moisture	1.03%
5.	" Acid insoluble	7.14%

8. Allow to stand for 30 min.
9. Filter the ppt and transfer to a preweighed crucible.
10. Ignite the ppt at 500°C till constant weight is obtained.

Calculations:

$$\frac{\text{Gain in wt (gm) x132}}{\text{wt of sample (gm)}} = \% \text{ Calcium as } Ca(OH)_2$$

$$\frac{\text{Gain in wt (gm) x100}}{\text{wt of sample (gm)}} = \% \text{ Calcium as CaO}$$

Determination of Acid Insoluble Residue

1. Weigh 1 gm dried sample into a 250-400 ml beaker and stir with 40 ml water.
2. Add 10 ml concentrated HCl and stir.
3. Warm the mix and boil for 10 minutes by adding constantly water to maintain the level.
4. Filter the content through Whatman # 40 filter paper.
5. Rinse the paper first with 30 ml hot distilled water and then with 30 ml 2N sodium carbonate solution.
6. Boil the contents of the beaker for 10 minutes.
7. Filter through the same paper (Whitman # 40).
8. Wash the residue 5 times with hot distilled water two times with 2N HCl and finally with hot distilled water till the filtrate is free from chloride.
9. Dry and ignite the residue at 500°C.

10. Cool and weigh the residue as Acid Insoluble Residue.

Determination of Moisture

1. Weigh 10 gms of lime in a pre-heated and weighed evaporating dish.
2. Keep the dish and lime at 103°C for 4 hrs.
3. Cool the sample and weigh.
4. Again keep it at 103°C for 1 hr, cool and weigh.
5. Repeat Step # 4 till constant weight is obtained.
6. Calculate % Moisture $(Y-Z) \times \frac{100}{S}$

   Y = weight in gms of dish + sample before heating

   Z = weight in gms of dish + sample after heating

   S = weight in gms of lime sample.

Determination of Sodium and Magnesium

1. Prepare an acid extract of the sample by dissolving 10 gms of sample in hot distilled water and 10 ml of concentrated HCl.
2. Filter through Whatman # 40 filter paper and copiously wash with hot distilled water.
3. Make up the volume to 100 ml.
4. The soil specimen is tested in a Perkin-Elmer 4000 Atomic Absorption Spectrophotometer for sodium and magnesium.
5. Sodium was tested using Emission mode and magnesium in a flam-eter's graphite furnace.

6.3 Effect of Chemicals on Soil Properties

6.3.1 Effect of Chemicals on Double Layer, Mineral Composition, Soil Fabric, CEC & pH

A portion of soil was taken from the soil samples already treated with chemicals and tested for swell pressure. Samples of required sizes were prepared for determination of mineral composition and identification of soil fabric. Soil samples in solution form were used to determine CEC and pH.

Before discussing the effect of stabilizing agents on chemical properties, it is necessary to know the effect of these chemicals on double layer. Following equation is the basis for any change in chemical properties of the soil mass after treating it with lime.

$$\underset{\text{Lime}}{Ca(OH)_2} + \underset{\text{Na-Montmorillonite}}{Na_2(Al,Mg)_2Si_4O_{10}(OH)_2 \cdot xH_2O} = \underset{\text{pH=14}}{2NaOH} + \underset{\text{Ca-Montmorillonite}}{Ca(Al,Mg)_2Si_4O_{10}(OH)_2 \cdot xH_2O} \qquad (6.5)$$

Expansive soils with Na-montmorillonite contain water molecules (H_2O) and Na^+ ions in the diffuse double layer. When lime is added to such soil samples calcium (Ca^{2+}) replace sodium ions (Na^+). This is because calcium is stronger than sodium due to its higher valency. As a result more stable Ca-montmorillonite compound will be formed. This is the reason why Ca-montmorillonite is less expanding than Na-montmorillonite. In case of Ca-montmorillonite, the interlayer

attraction arising from Van der Waals forces will be stronger and more resistance will be offered to interlayer hydration than in the case of Na-montmorillonite [9].

The compositions of soil samples after treatment with commercial lime, potassium chloride and potassium phosphate are shown in Table 6.2. The mineralogical composition was found using x-ray diffraction method as described earlier. As shown in Table 4.1, soil sample from block # 03 with no treatment contains upto 23% Na-montmorillonite. As the soil sample is treated with different chemicals, the amount of Na-montmorillonite becomes almost negligible and is converted into less expanding clay & non-clay minerals as listed in Table 6.2. The new minerals formed are Ca-montmorillonite, illite, calcite, sylvite, albite and mixtures of potassium and magnesium salts. Although Ca-montmorillonite and illite fall in the category of expanding clay minerals but as explained in chapter 2, those are less expansive in nature. Calcite and sylvite are simply calcium and potassium salts which are non expansive. Similarly albite and other K-Mg salts help in reducing the plasticity of swelling soils.

SEM photomicrographs for samples from block # 03 treated with stabilizing agents are shown in Figs. 6.2 to 6.4. Fig. 6.2(a) and (b) are photomicrographs for soil samples with 5% commercial lime. Comparing these photographs with Figs. 4.10(a) & (b) for soil sample from block # 03 without reagent, it is observed that the fabric both along and perpendicular to layers are quite different from each

Table 6.2 : X-Ray Diffraction Results for Treated Soil Samples

Minerals	Fraction
SAMPLE FROM BLOCK # 03 + 5% COMMERCIAL LIME	
Quartz SiO_2	57.5%
Na-Montmorillonite $Na_{0.3}(Al,Mg)_2Si_4O_{10}(OH)_2.X\ H_2O$	2.0%
Ca-Montmorillonite $Ca(Al,Mg)_2Si_4O_{10}(OH)_2.X\ H_2O$	10.0%
Illite $K_{0.7}Al_{2.1}(Si,Al)_4O_{10}(OH)_2$	25.0%
Calcite $CaCO_3$	5.5%
SAMPLE FROM BLOCK # 03 + 5% POTASSIUM CHLORIDE	
Quartz SiO_2	45%
Sylvite KCl	30%
Illite $K_{0.7}Al_{2.1}(Si,Al)_4O_{10}(OH)_2$	23%
$KMg_{2.5}Si_4O_{10}(OH)_2$	2%
SAMPLE FROM BLOCK # 03 + 5% POTASSIUM PHOSPHATE	
Quartz SiO_2	59.0%
Albite Potassiam High $(Na,K)Al\ Si_3O_8$	10.5%
Illite $K_{0.7}Al_{2.1}(Si,Al)_4O_{10}(OH)_2$	27.5%
$KMg_{2.5}Si_4O_{10}(OH)_2$	3.0%

Fig. 6.2(a)

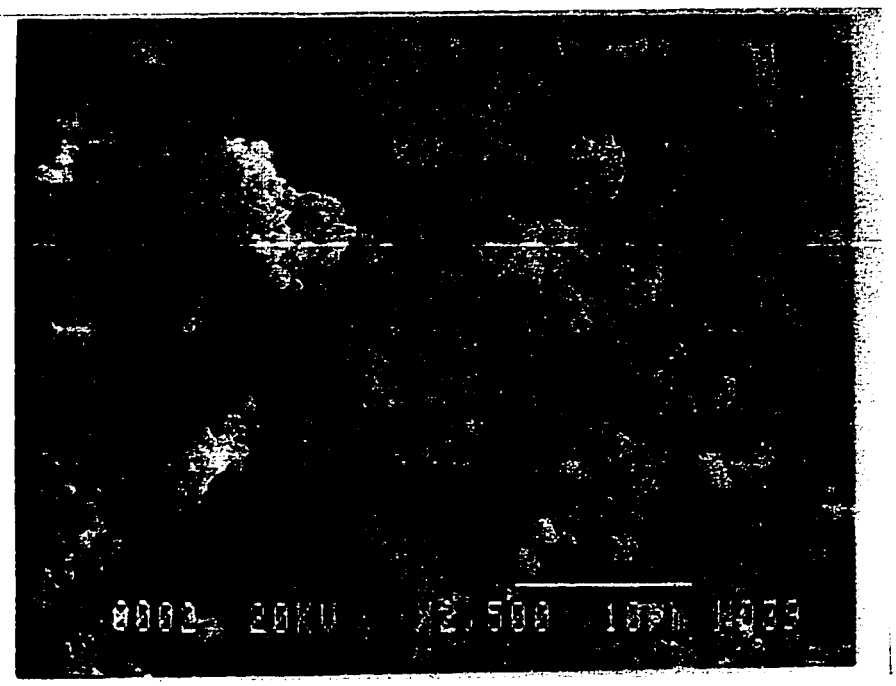


Fig. 6.2(b)

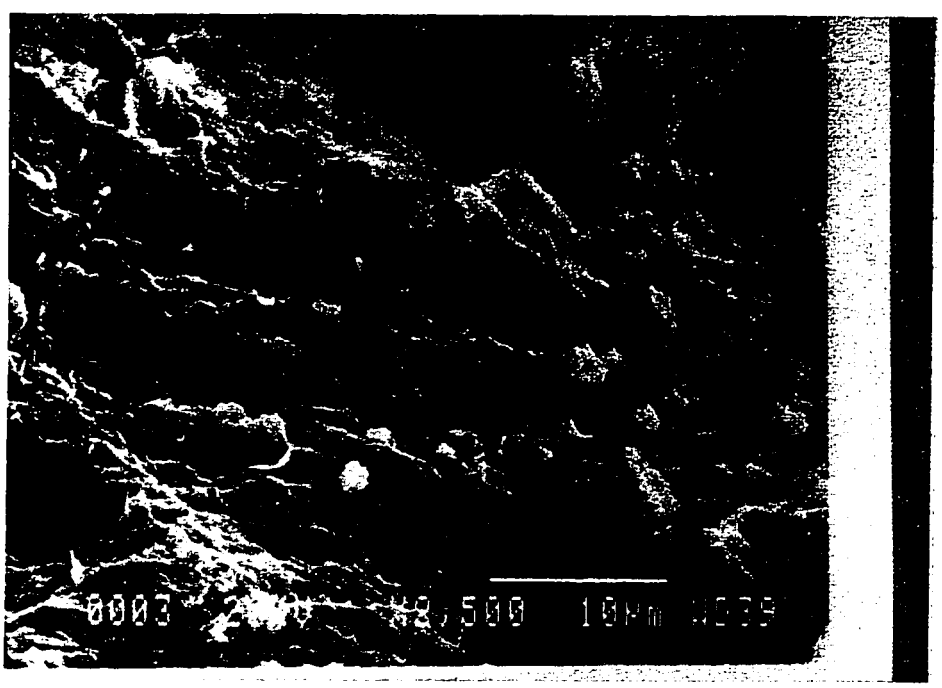


Fig. 6.2 : SEM Photomicrographs for Soil Samples from Block # 03 treated with 5% commercial lime.
(Upper photograph showing fabric along layers, lower one perpendicular to layers).

Fig. 6.3(a)

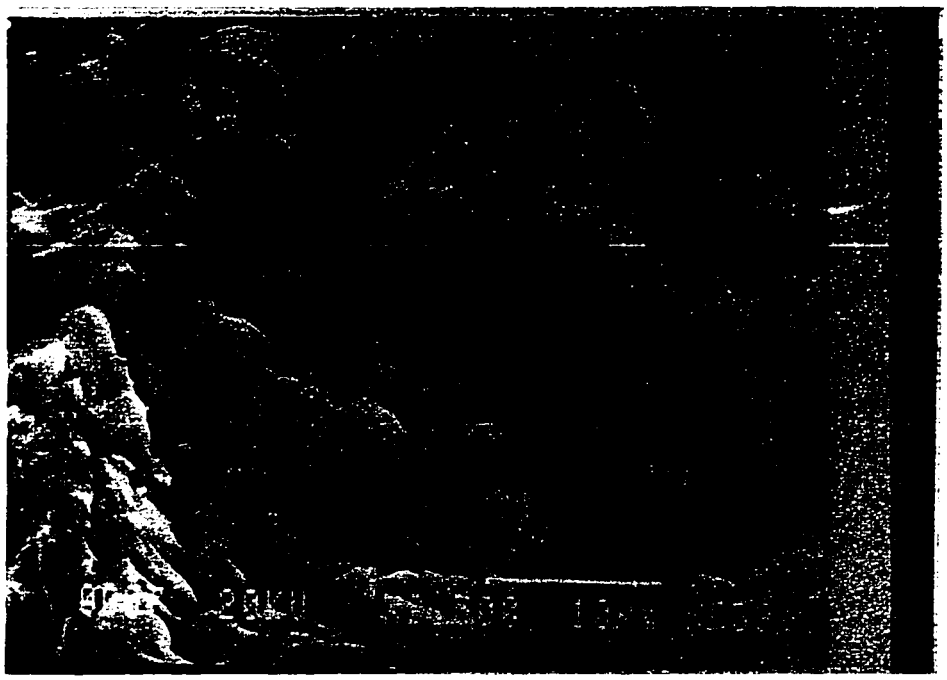


Fig. 6.3(b)

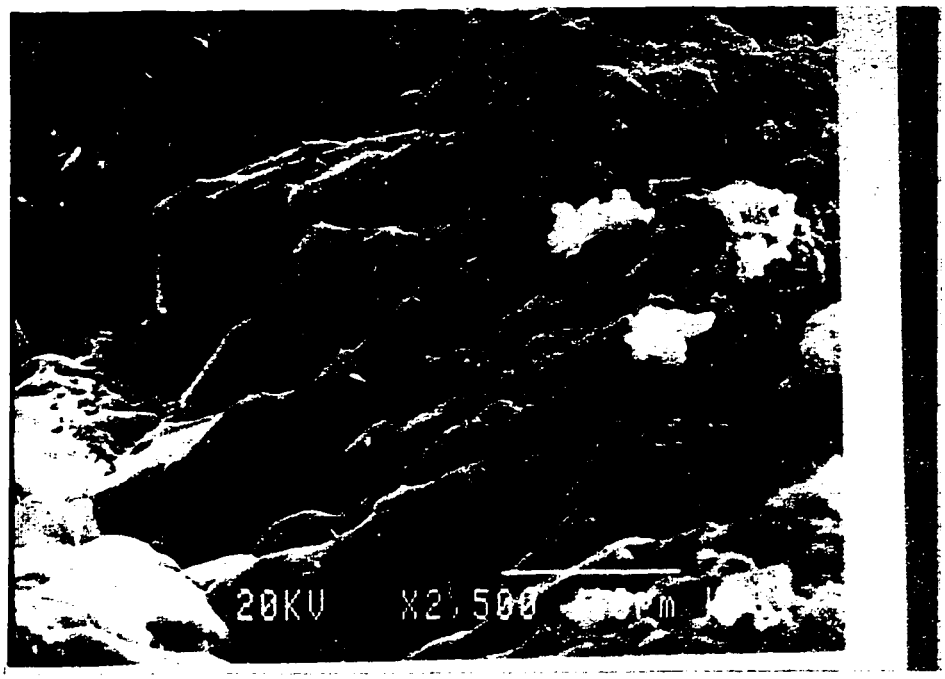


Fig. 6.3 : SEM Photomicrographs for Soil Samples from Block # 03 treated with 5% potassium chloride.
(Upper photograph showing the fabric along layers, lower one perpendicular to layers).

Fig. 6.4(a)

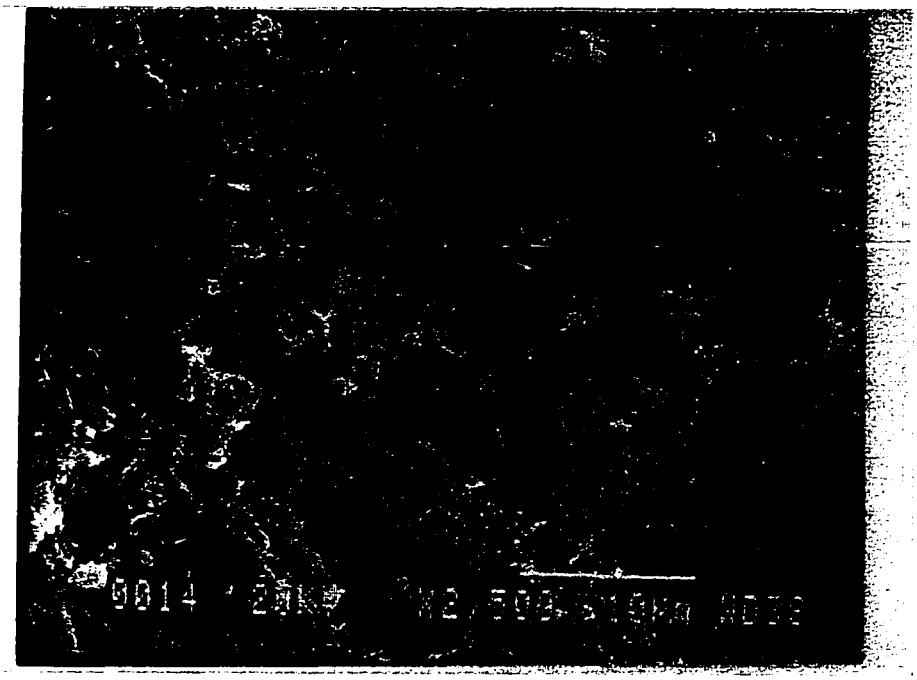


Fig. 6.4(b)

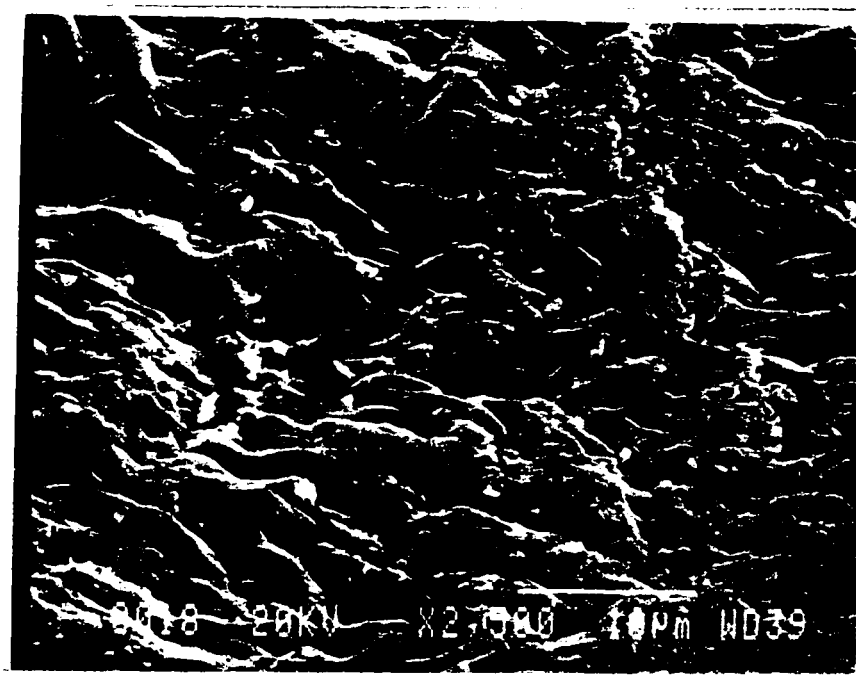


Fig. 6.4 : SEM Photomicrographs for Soil Samples from Block # 03 treated with 5% potassium phosphate.
(Upper photograph showing fabric along layers, lower one perpendicular to layers).

other. The white spots in the photographs represent calcite ($CaCO_3$) mineral. These white spots are in predominance only in Figs. 6.2(a) and (b) due to addition of commercial lime. Figs. 6.3 & 6.4 show photomicrographs for soil samples from block # 03 treated with 5% potassium chloride and 5% potassium phosphate respectively. Since these two chemicals were used in solution form, not much spots are observed. Although much changes in mineralogical composition take place as reflected by Table 6.2, but not a single mineral can be seen clearly from the photomicrographs in Figs. 6.3, only a coating of the chemical is observed. In Figs. 6.4, for sample with 5% potassium phosphate, the white spots are of potassium high compound. Although the situation is not very clear but it can be concluded that there is a change in the fabric due to treatment with chemicals.

Table 6.3 summarizes the values of cation exchange capacities and pH. These two parameters were determined by standard procedure as discussed in Chapter 4. A striking feature of all the three chemicals is that the cation exchange capacity reduces on increasing the chemical content while pH increases. Usually pH is directly proportional to cation exchange capacity. Again the answer to this question is Eq. (6.5) where Ca-montmorillonite is formed on addition of lime. Since this compound (Ca-montmorillonite) is more stable than Na-montmorillonite, not much exchange of ions takes place after pozzolanic reaction is completed. Equations similar to (6.5) can be obtained for potassium chloride and potassium phosphate where

Table 6.3 : Cation Exchange Capacity & pH for Soil Samples from Block # 03 Before & After Treatment

Sample Type	CEC (meq./100 gm)	pH
Soil sample + no reagent	24.09	7.91
Soil sample + 2% commercial lime	20.46	9.92
Soil sample + 5% commercial lime	18.90	9.97
Soil sample + 8% commercial lime	15.82	9.98
Soil sample + 2% potassium chloride	21.89	10.10
Soil sample + 5% potassium chloride	19.08	10.13
Soil sample + 8% potassium chloride	17.56	10.15
Soil sample + 2% potassium phosphate	21.50	9.90
Soil sample + 5% potassium phosphate	19.01	9.96
Soil sample + 8% potassium phosphate	15.99	9.97

K-montmorillonite is formed instead of Ca-montmorillonite. Again K-montmorillonite is more stable than Na-montmorillonite hence there will again be a reduction in CEC values on addition of these chemicals while of course the pH will increase at a higher rate than in case of commercial lime as shown in Table 6.3.

As shown in Eq. 6.5, another compound formed is NaOH which has a pH value of 14 when found in pure form. But since NaOH is one of the resulting compounds, the pH of the mixture is less than 14. As we increase the lime content the amount of NaOH increases thereby increasing pH of the whole mixture. At the same time more stable compounds are formed as we increase the lime content and hence CEC of the mixture should decrease as shown in Table 6.3. A comparison can be made between the reactivity, stability and CEC of the three types of cations Ca^{+2}, K^{+} and Na^{+} as described in Table 6.4. [12]. Results recorded in Table 6.3 confirm this rating.

6.3.2 Effect of Commercial Lime on Index Properties

Four percentages (2,4,6 and 8%) of commercial lime were added to soil samples from block # 03. The mixtures were kept for about a week so that soil-lime reaction would be effective. The liquid and plastic limits were determined as discussed earlier. Test results are shown in Fig. 6.5. The P.I. curve is drawn simply by subtracting L.L. and P.L. curve ordinates at different percentages of lime.

As the percentage of lime is increased from 2 to 4%, there is a

Table 6.4 : Comparison between Reactivity, Stability & Cation Exchange Capacities of three types of cations (Ref. 12)

Cations	Compound	Reactivity	Stability	cec
Ca^{2+}	$Ca(OH)_2$	Lowest	Highest	Lowest
K^+	KOH	Medium	Medium	Medium
Na^+	NaOH	Highest	Lowest	Highest

Table 6.5 : Effect of Lime-Treatment on Index Properties of Soil Samples from Block # 03

Commercial Lime (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index
0	152.8	42.9	109.9
2	152.2	56.4	95.8
4	147.6	104.1	43.5
6	133.8	112.8	21.0
8	126.2	118.5	7.7

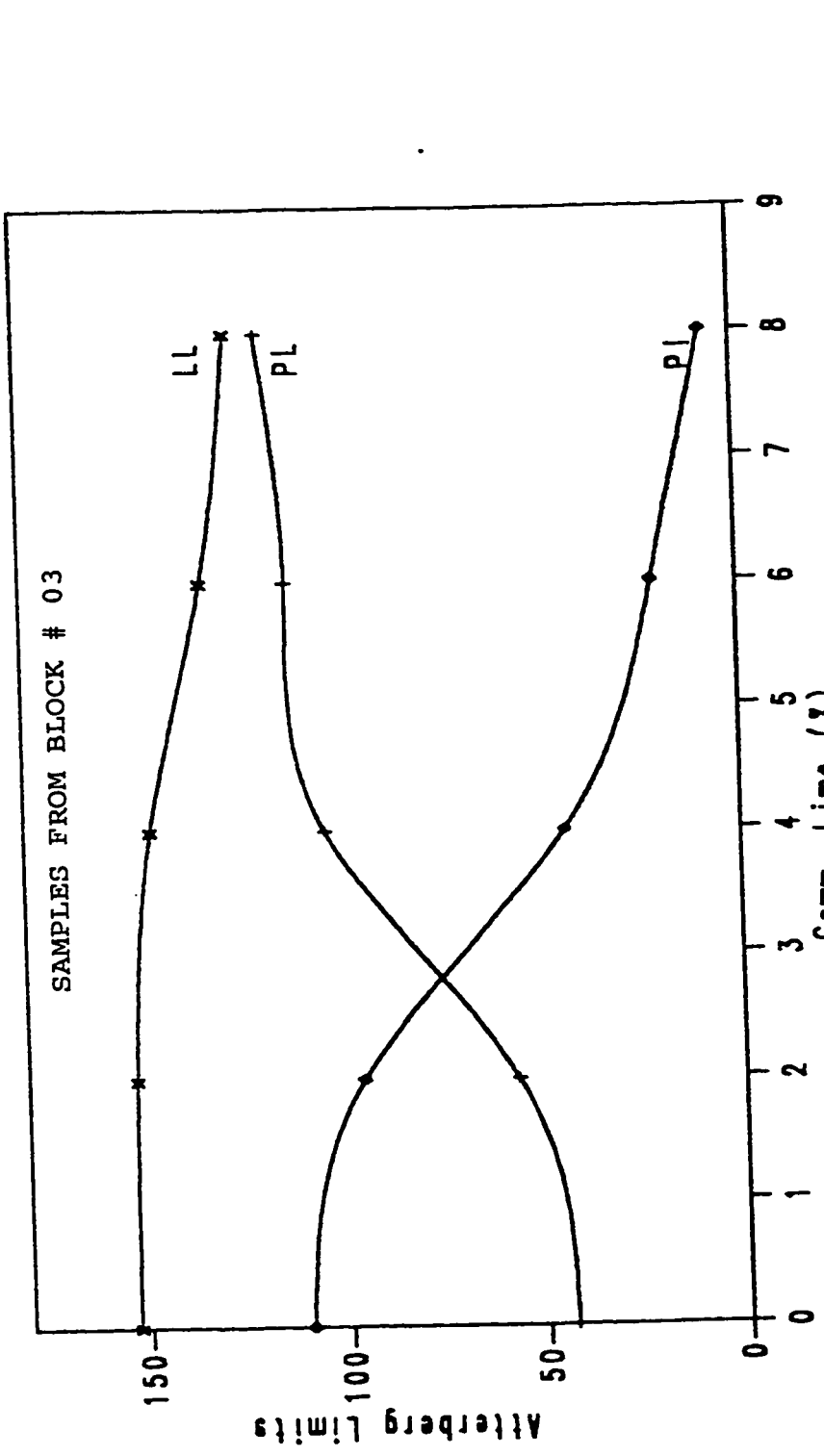


Fig. 6.5 : Percentage Commercial Lime versus Atterberg Limits

sudden change in L.L., P.L., and P.I. The addition of commercial lime to the wet soil samples decreases the P.I. from 109.9% for untreated soil to 7.7% for the soil sample treated with 8% lime as shown in Table 6.5. The soil becomes almost non-plastic at higher lime percentages. The major reduction in plasticity was achieved when 4-8% of commercial lime was used. So, this lime content should be chosen for treatment of soil mass since this amount effectively decreases plasticity. Comparing P.I. values in Table 6.5 with those given in Table 2.3, we see that the untreated sample with P.I. as 109.9% fall in the category of very high swelling soil while the same sample treated with 8% lime has P.I. value of only 7.7% and thus becomes non-expansive. Lime content also affects workability. The silty and fragile texture of the treated soil causes a marked decrease in workability. The improved level of workability expedites subsequent manipulation and placement of the treated soil mass [30].

6.3.3 Effect of Commercial Lime on Dry Density After Compaction

A portion of the soil block # 03 was broken into pieces and standard compaction test was performed to get the moisture-density curve as shown in Fig. 6.6. Three different percentages of commercial lime 2%, 5% and 8% (by dry weight of soil) were added to soil samples. The soil-lime mixtures were mixed thoroughly. The entire mixing process was done in a sealed plastic bag and Standard Compaction test was performed after one week curing of the soil samples.

Fig. 6.6 shows that the optimum moisture content and maximum

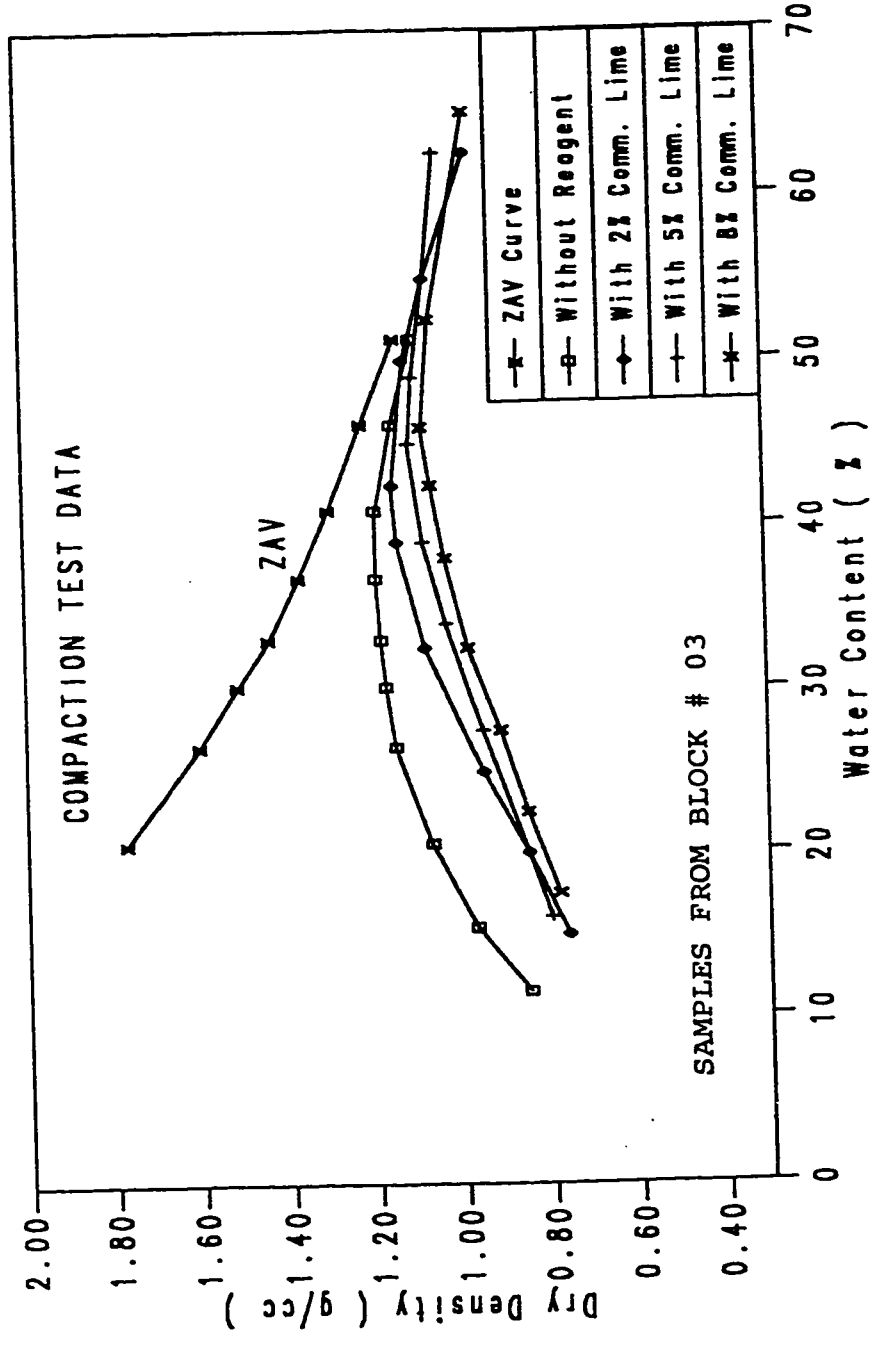


Fig. 6.6 : Water Content versus Dry Density Curves for Soil Samples from Block # 03 with Different Percentages of Commercial Lime

dry density vary with different percentages of commercial lime. Maximum dry density decreases and optimum moisture content increases as the percentage of lime increases. Increasing lime content will produce more flocculate arrangement of particles which cause a reduction in dry density [31].

6.4 Laboratory Evaluation of Chemical Stabilization of Al-Qatif Clays

Samples from block # 03 were used for chemical stabilization because this block was identified as most expansive among all others collected from Al-Qatif. Potassium chloride, potassium phosphate, pure lime and commercial lime were used for determining the effect of these chemicals on swell pressure.

6.4.1 Stabilization with Commercial Lime and Pure Lime

Different percentages of commercial lime and the broken pieces of wet soil from block # 03 were mixed thoroughly and placed in a mold having inner diameter equal to the internal diameter of the consolidometer ring. The mold with sample in it, was then placed on the platform of a versa tester for static compaction. The loose soil-lime mixture was compacted to a thicknesss equal to the thickness of the metal ring. After compaction to the required density, the sample was taken out of the mold carefully and placed in the consolidometer ring. The sample was kept in plastic cover to avoid any loss of natural moisture content. So, the sample was ready for testing. After setting up the apparatus for swell pressure test, the sample was

unwrapped and put in the consolidometer. Swelling pressure test was used for measurement of swell pressure of soil samples. Each test was continued for about a week.

Lime flocculates the clay minerals due to base exchange and its application causes cohesive soils to become more workable and less plastic; it can also reduce swelling and shrinking due to gain or loss of moisture [30]. The value of swell pressure was compared with that of untreated soil sample. A marked reduction in swell pressure was observed with 8% commercial lime. The results have been plotted as shown in Fig. 6.7. Soil specimen with 8% commercial lime exhibit high swell pressure values in the beginning. High percentage of lime will increase permeability which causes faster swelling initially. But due to reduction in CEC values (as reported in Table 6.3) and formation of less expanding clay minerals (as shown in Table 6.2) on addition of commercial lime, the final swell pressure reduces as the percentage of commercial lime is increased.

Fig. 6.8 shows the effect of curing time on swelling pressure. 5% of commercial lime was used for that purpose. Not much difference was observed between 7 days and 14 days curing. This means that the soil-lime reaction is very effective only for about a week after the application of commercial lime. So, the site where soil is to be treated with commercial lime, should be kept untouched for one to two weeks to achieve proper stabilization. Construction of any form shall proceed after this period to insure safety.

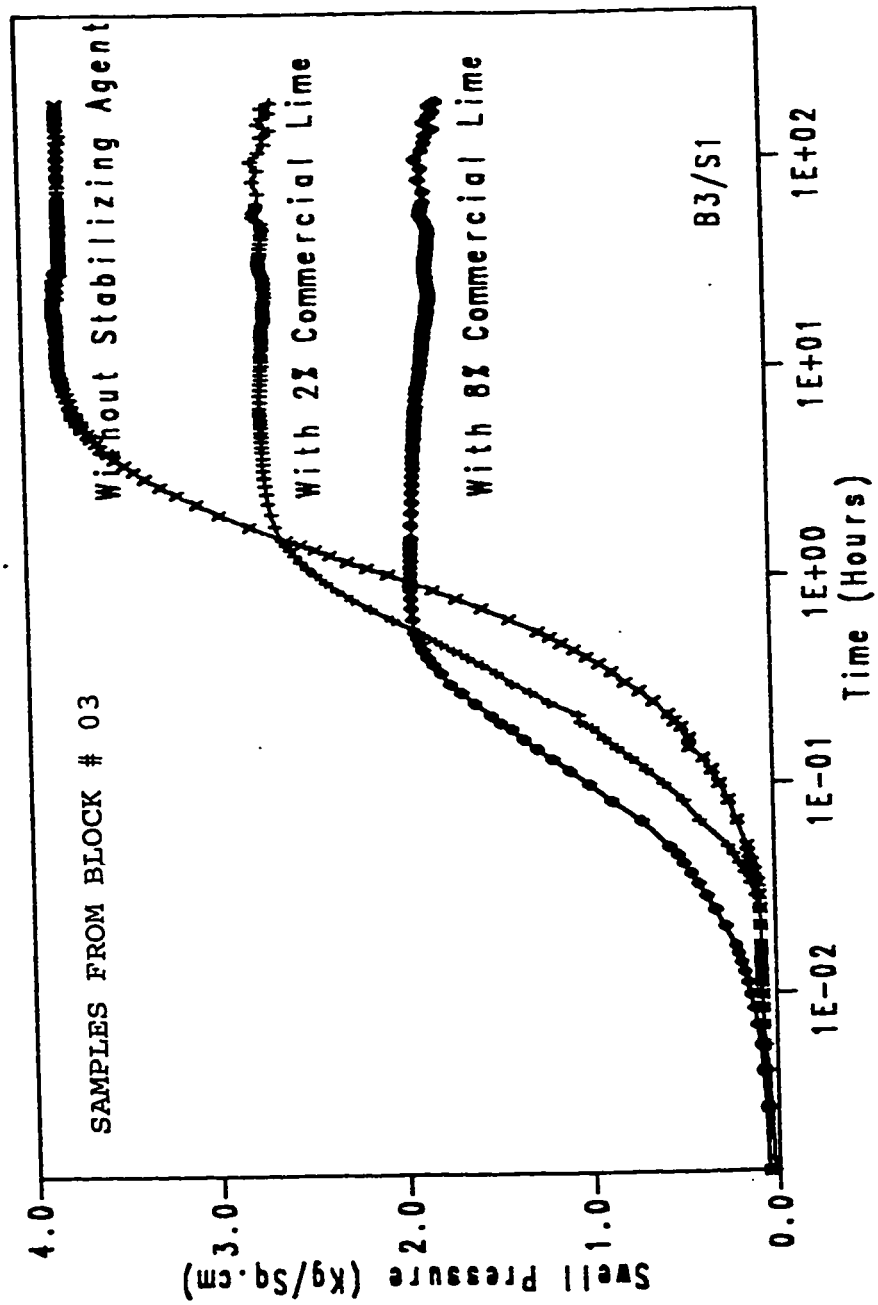


Fig. 6.7 : Effect of Commercial Lime on Swell Pressure

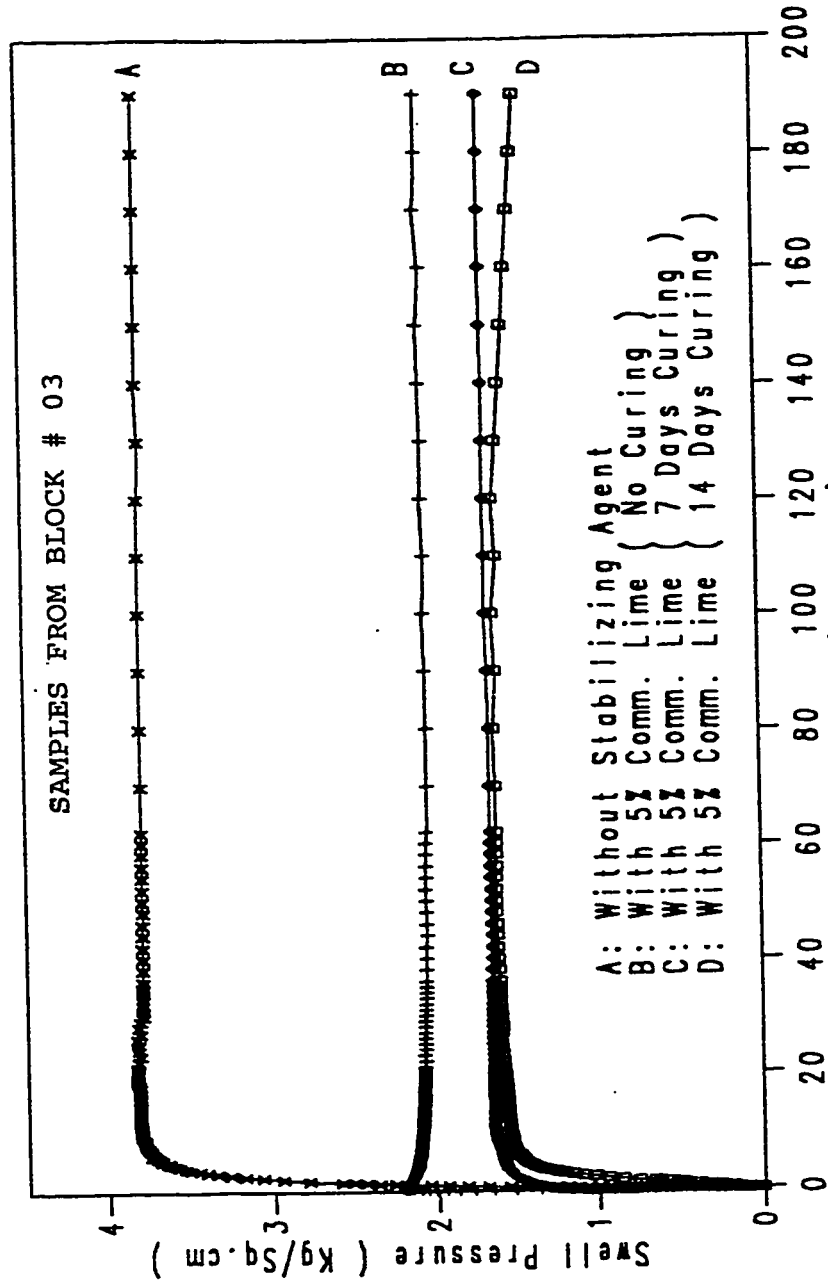


Fig. 6.8 : Effect of Curing with 5% Commercial Lime on Swell Pressure

Fig. 6.9 shows the effect of pure lime & commercial lime on swelling pressure. Soil samples with 5% of the two types of limes were tested in the odeometer and the swell pressures were calculated with the help of the data logger. Since pure lime can dissolve in water, so it was used with water in the consolidometer while commercial lime was mixed with broken pieces of soil. It was observed that 5% commercial lime was more effective in reducing the swell pressure as compared to the same percentage of pure lime. Commercial lime exhibits better results because of the impurities listed in Table 6.1. These impurities are nothing but salts of magnesium & sodium which also react with the soil & help in reducing its swelling pressure. So, the use of commercial lime can be regarded as lime-salt treatment which has more pronounced effect on swelling pressure than the pure lime as depicted in Fig. 6.9. Another advantage of using commercial lime for stabilization is its low cost which makes it economically attractive. A 50 Kg. bag of commercial lime costs around SR.10 while we can get only 50 g of pure lime with this amount of money.

6.4.2 Stabilization with Potassium Chloride and Potassium Phosphate

The use of other compounds such as potassium phosphate, Fig. 6.10, and potassium chloride, Fig. 6.11, has also been investigated in the present research on Al-Qatif clays. These chemicals were used in solution form since they are soluble in water. The compounds were added with water directly in the consolidometer. The soil samples were tested in the disturbed state as in the case of com-

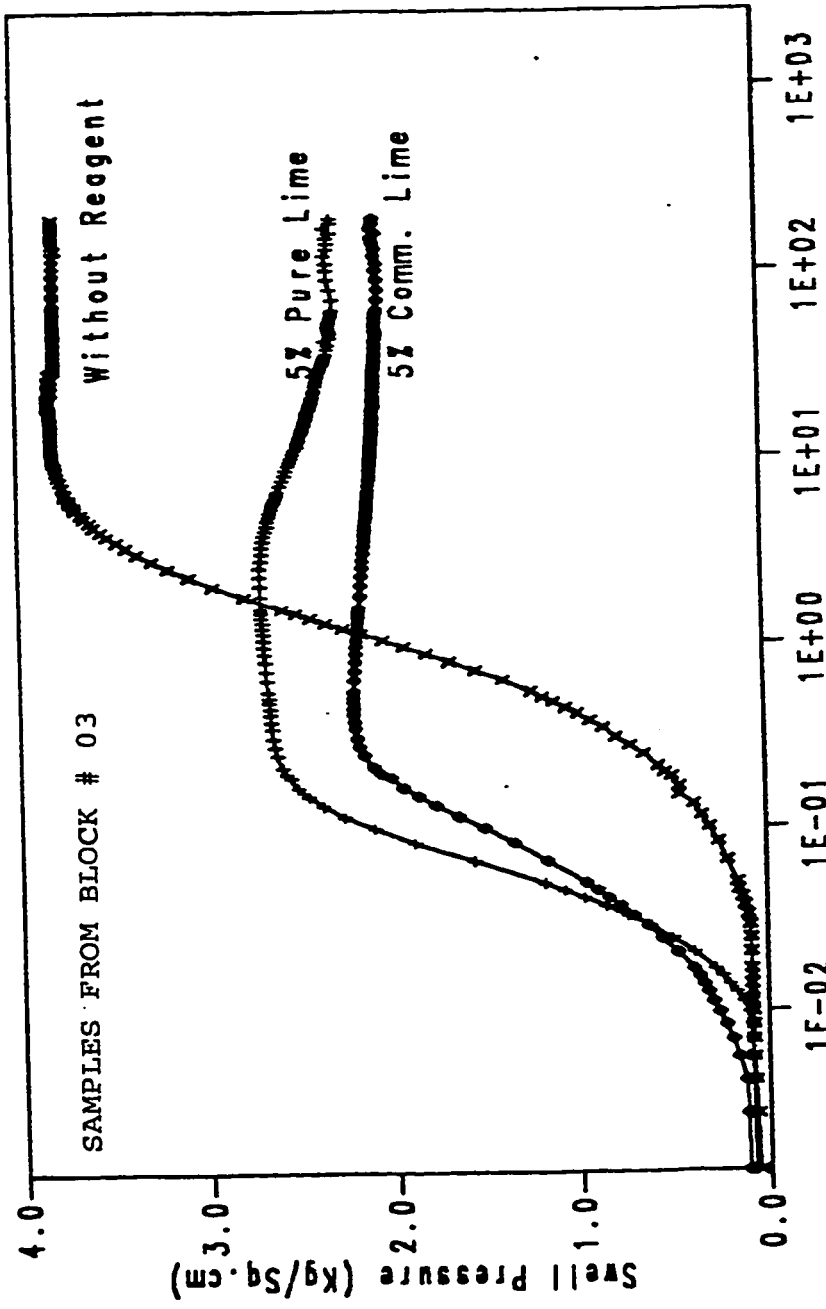


Fig. 6.9 : Comparison of Pure Lime and Commercial Lime as Stabilizers

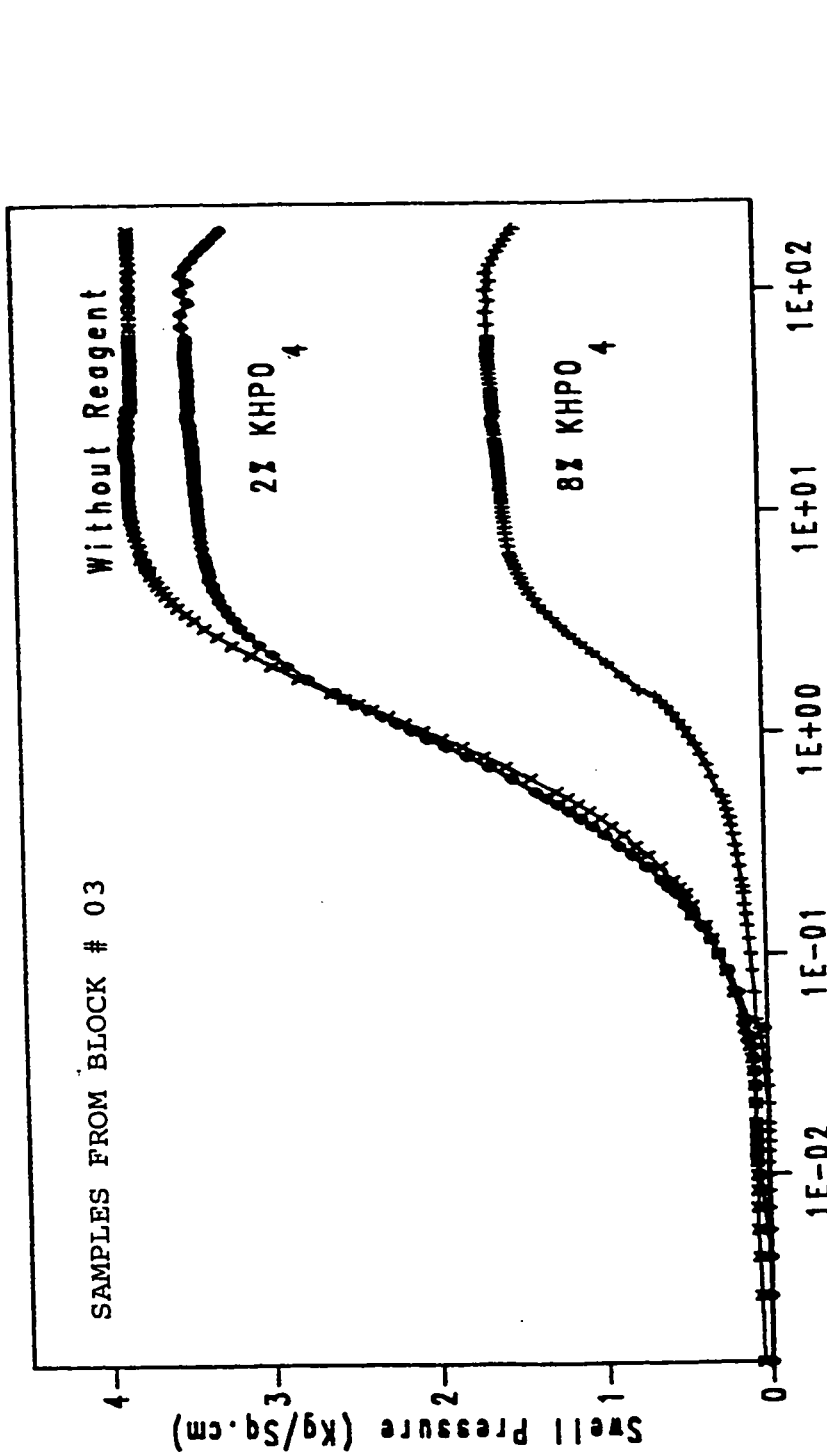


Fig. 6.10 : Effect of Potassium Phosphate on Swell Pressure

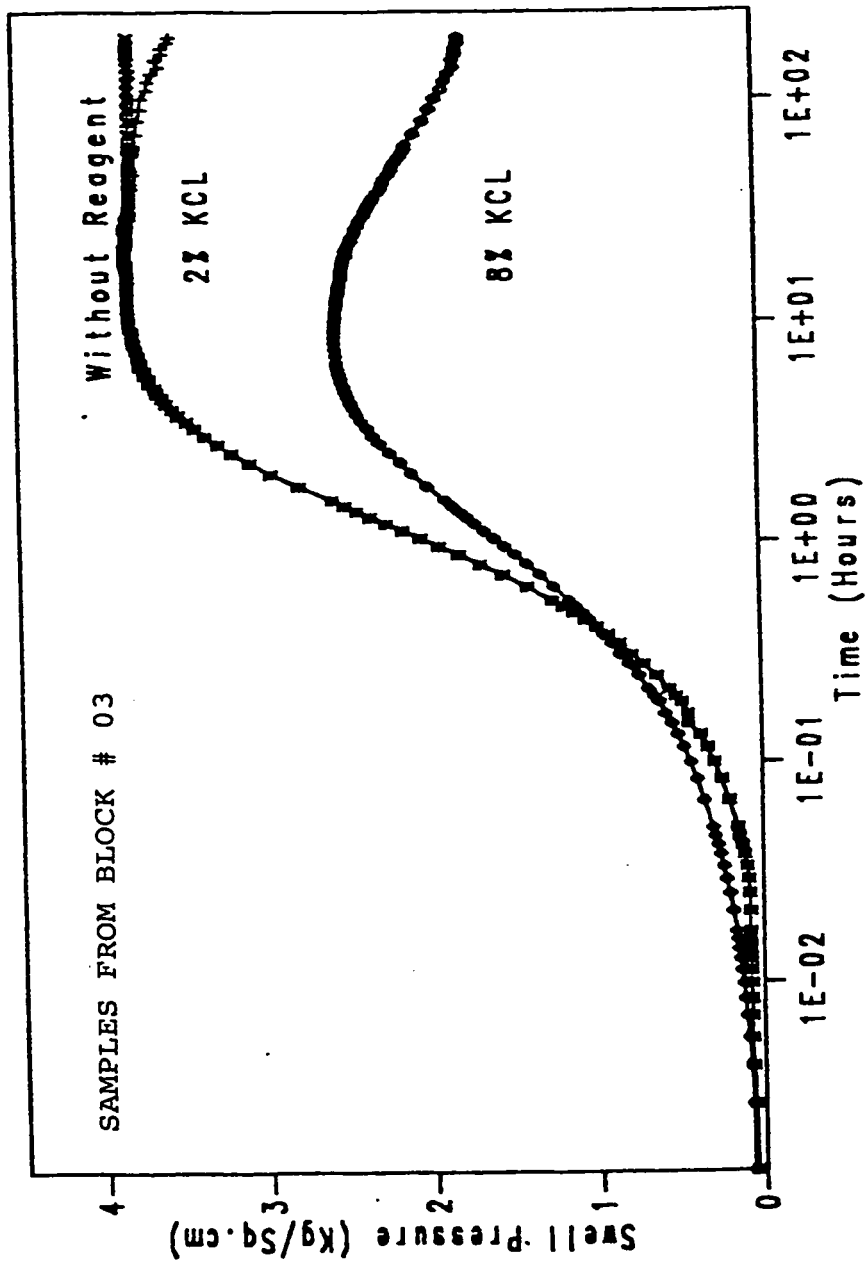


Fig. 6.11 : Effect of Potassium Chloride on Swell Pressure

mercial lime. Results showed that additions of these compounds to expansive clays caused a significant decrease in swelling pressure and apparently caused mineralogical changes linked with soil stabilization. It can be observed from Figs. 6.10 and 6.11 that the reaction of these compounds with soil is more pronounced after about a week. This is verified from the fact that after 3 or 4 days, the curves stabilize and a sudden drop is observed after about a week or so. This means that the soil if treated with these additives should be kept for some more time for proper stabilization.

In Fig. 6.12 a comparison is made between the same percentages of potassium chloride and potassium phosphate. It is observed that $KHPO_4$ is more effective in reducing swell pressure. Fig. 6.13 shows curves joining points of various swell pressure values for different percentages of reagents. It can be inferred from Fig. 6.13 that commercial lime (Curve C) can be regarded as the best stabilizing agent among all four kinds of compounds studied. This is because it can reduce swell pressure values to a greater extent even at low percent additions. Then comes potassium phosphate and lastly potassium chloride.

The same data of Fig. 6.13 are again shown in Fig. 6.14 where a comparison is made between swell pressure values for soil specimens treated with three types of stabilizing agents. This comparison by histograms also indicate that values of swell pressure are lower for commercial lime as compared to other reagents even for 2% of the

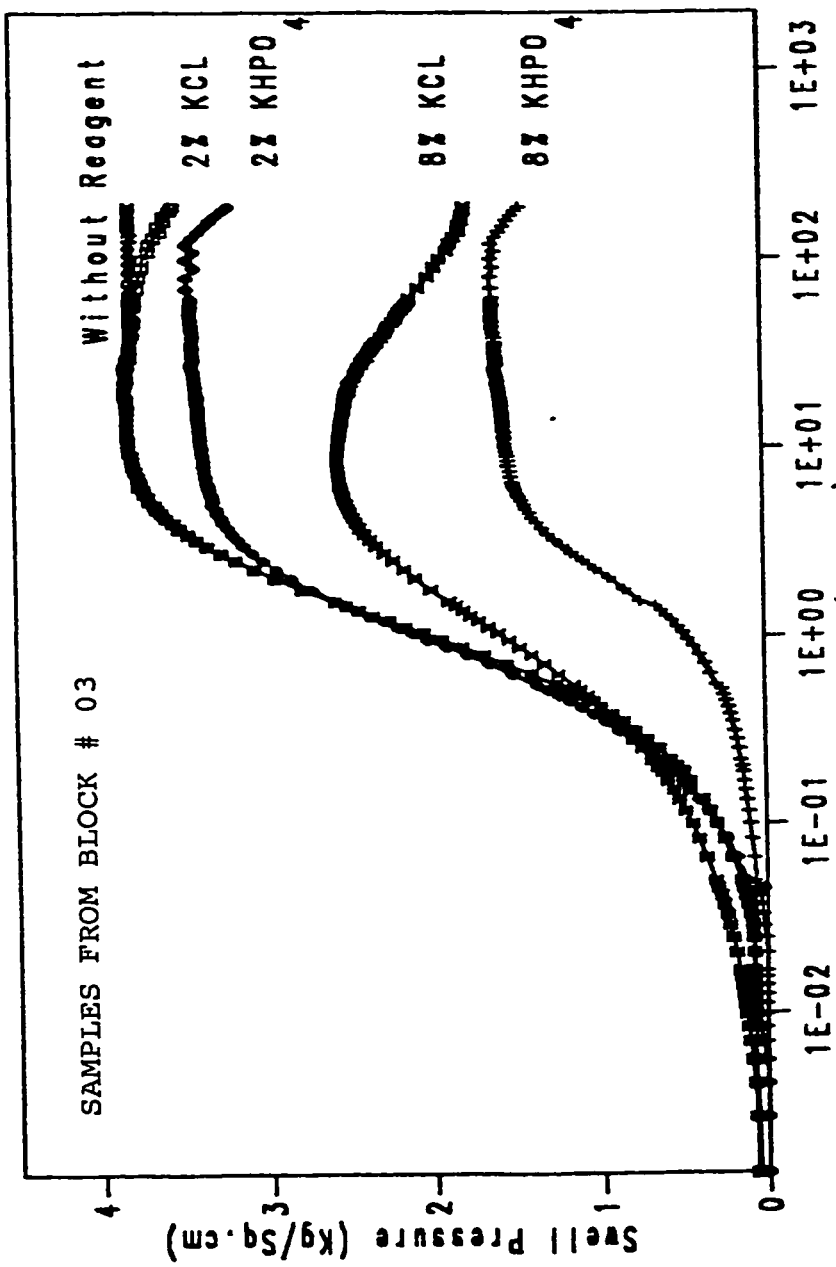


Fig. 6.12 : Comparison of KCl and $KHPO_4$ as Stabilizing Agents

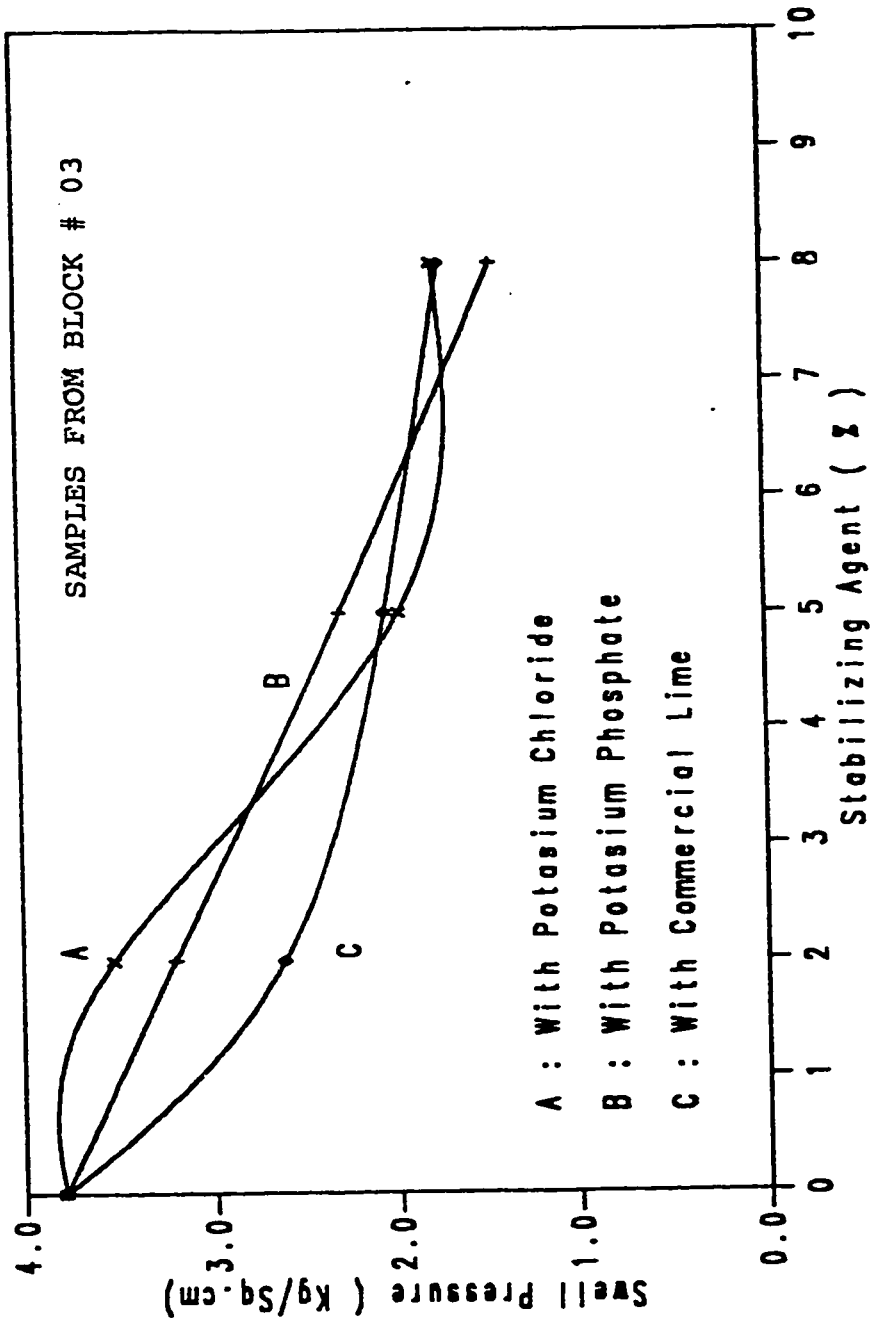


Fig. 6.13 : Curves Showing Variation in Swell Pressure Values for Different Percentages of Reagents

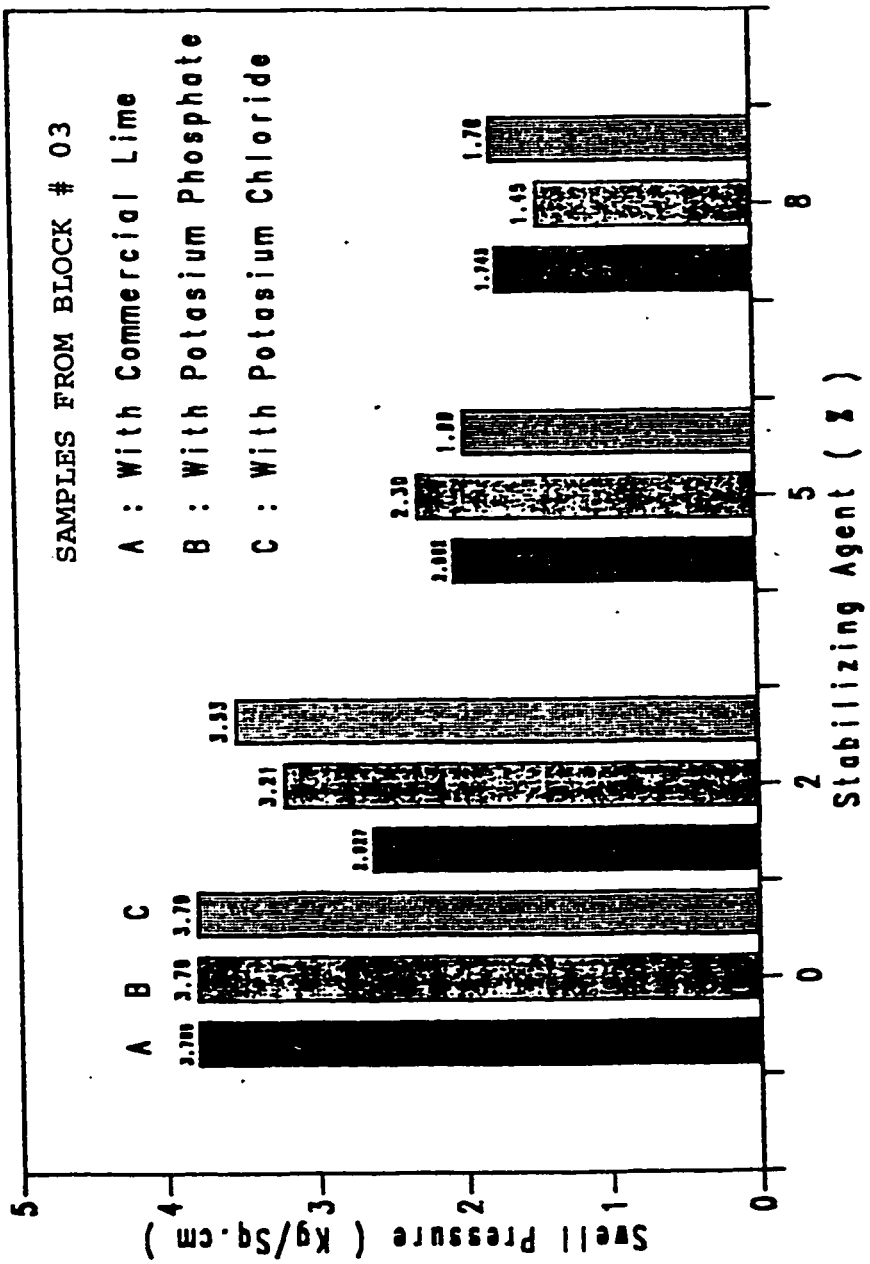


Fig. 6.14 : Comparison of Swell Pressure Values for Different Percentages of Stabilizers

stabilizers. The histograms in Fig. 6.14 also show that there is not much difference in the values of swell pressure as we increase the percentage of any reagent from 5 to 8%. It can be predicted from these results that only 6% of any kind of compound will be sufficient for proper soil-stabilization.

Table 6.6 summarizes the maximum swell pressure values corresponding to various percentage of the three types of stabilizing agents used in this study for laboratory stabilization of Al-Qatif expansive clays.

Table 6.6 : Swell Pressure Values for Soil Samples from Block # 03 with Various Percentages of Stabilizing Agents

Percent Reagent	Swell Pressure (Kg/cm^2)		
	Commercial Lime	Potassium Chloride	Potassium Phosphate
0%	3.79	3.79	3.79
2%	2.03	3.53	3.21
5%	2.00	1.99	2.30
8%	1.74	1.78	1.45

CHAPTER 7

MEASURES AGAINST HEAVE

7.1 Preconstruction Soil Treatment

In trying to prevent or mitigate damage caused by expansive soils the best way is to take all precautionary measures before constructing any type of structure. Preconstruction soil treatment options can be categorized into three groups [15]:

(1) reworking the soil, moving it and controlling its density

(2) controlling its water content

(3) adding chemical stabilizers to it.

Reworking Soil

Soil-reworking options may include:

* Replacement of an expansive soil layer by a non-expansive low permeability soil like clayey silts. The method should be followed when the uppermost 3 ft or less is highly expansive.
* The previous option plus reworking and moisture control of deeper soils under the building. The soil is reworked to reduce its density, lessening its swell potential. Soil moisture is controlled at or slightly above optimum, which is near saturation and thus near soil's maximum probable swell in the years after construction.

* Intermixing or blending expansive and non-expansive soils, reducing montmorillonite (smectite) content to under 10%. This usually eliminates the potential for soil volume change resulting from changing soil moisture. Blending must be uniform & intimate [15].

Controlling Soil Moisture

The second class of options is to control the clay's water content. The goal is to keep water from moving into, or out of the soil under and near the building. If done successfully, this reduces or eliminates post-construction swelling or shrinking [7].

Generally, expansive clays should be placed and kept at moisture contents slightly higher than optimum for compaction to maximum dry density, according to ASTM D-698 procedure. Since heave rarely occurs after a soil mass is saturated, moisture contents as recommended usually prevent heave damage, but potential shrinkage must be controlled. Where there is concern about potential moisture loss and resultant soil shrinkage, one solution is to deepen building foundation to the approximate depth of the active zone. This is particularly necessary if construction takes place in the springs, when soils are wetted to considerable depth but thereafter will be subjected to drying during long, hot summers [8].

Soil moisture can be controlled during construction or subsequently. Among the ways to control soil moisture are [15]:

1) Surface flooding or sprinkler irrigation. Expansive clays tend to be very fine grained, and thus low in permeability, so saturating them to depth may take considerable time. Frequently, surface wetting leaves the surface soils oversaturated and unworkable even though water penetrates only moderately into the active zone. The technique is generally unpredictable, uncontrollable and of limited value. But it is sometimes workable in soils that are only slightly to moderately expansive.
2) Another option is to keep in place moisture in the soil beneath the building by adding vertical and/or horizontal barriers. One horizontal barrier is the foundation and slab-on-grade itself. In addition, vertical or horizontal moisture barriers of plastic, rubber or even asphalt sheeting can be added.

Chemical Treatment

A third class of option as discussed in detail in Chapter 6, involves addition of chemical stabilizers to the expansive soils. This is usually done by specialized contractors. The approach should not be considered unless the contractor provides an acceptable, long-term site-performance guarantee backed by assured funding. However, a contractor's cost estimate for site stabilization is a useful yardstick against which to compare other site-development and construction options [30].

A number of chemicals can be used for stabilization of expansive soils but most of them are very expensive and are limited only

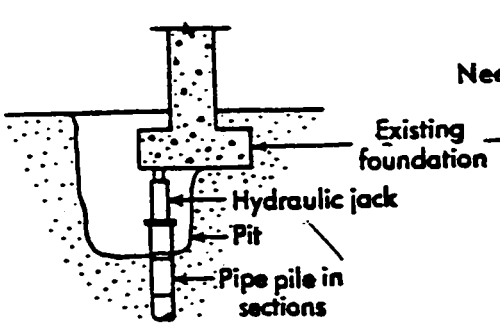
to laboratory testing. Commercial lime which has been discussed earlier, was found to be effective and very economical to use. In recent years lime-fly ash mixtures have also been used for road construction on expansive soils. In most of the industrialized nations, fly ash is a locally available material and if not properly reused it creates problems where it is stored [8].

7.2 Underpinning Existing Structures

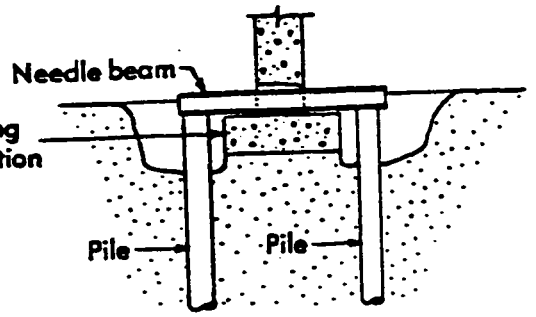
In some cases when expansion of underneath soil is detected at a later stage & progressive failure of superstructure starts, it becomes necessary to replace or strengthen the foundation of an existing structure. The operation of providing new permanent foundation is known as underpinning. This technique is used when the foundations of a structure prove incapable of supporting the structure with adequate safety or without undue settlement or shrinkage. Underpinning is also required when changing conditions, such as construction of a nearby building with a deep basement making existing foundations inadequate [2].

Two approaches are commonly followed: first, installing new foundations in small pits excavated beneath the existing foundations; and second, installing new foundations adjacent to the old and transferring the load between new and old with steel beams. [27].

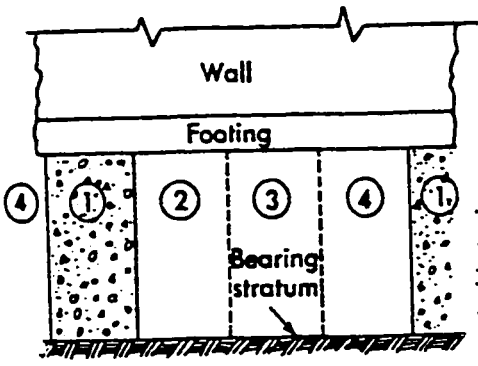
The pit method (Fig. 7.1a and c) requires excavating a small hole beneath part of the existing foundation. A new, deep footing is

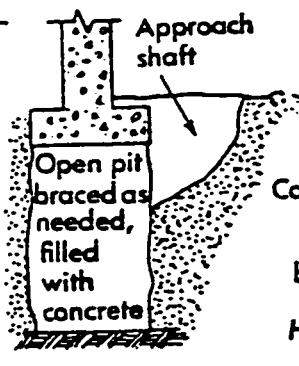


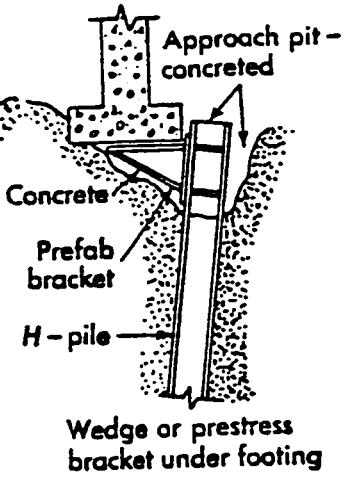
a. Pit method. Pipe piles driven by jacking against existing foundation



b. Needle beam method. Piles driven beside existing foundation







c. Pit underpinning of wall footing in sections

d. Bracket underpinning

Fig. 7.1 : Methods of Underpinning an Existing Foundation with Piles (Ref. 21)

poured in this hole, or piles are forced into the soil by jacking against the existing foundation. The pipes, in sections about 2 ft or 60 cm long, are jacked into the soil and then excavated by small buckets, a steam jet, or augers. The new foundation is placed section by section so that the old foundation is never without some support.

The second method (Fig. 7.1b) involves driving piles or constructing a new foundation as close as possible to the old. This is necessary when the old foundation is so small or weak that a pit beneath it is impossible. It is often cheaper than the pit method. The load is transferred from the old to the new foundation by horizontal needle beams which are placed beneath the old footing or through it. Clamps tightly bolted to a notched concrete or welded to a steel column make it possible to place the needle beam above the footing [21].

A bracket system, Fig. 7.1d employs a small drilled pier or pile reinforced to support bending moments. The pile or pier is placed immediately adjacent to the foundation and terminated about 2 ft or 60 cm below it. A bracket of steel or reinforced concrete is then constructed under the old foundation and either wedged or prestressed to pick up the load. Steel members are encased in concrete after prestressing, for corrosion protection.

The transfer of the load from the old to the new foundation is accompanied by some settlement. This may be eliminated by jacking

the new foundation against the old until the new carries the load. The settlement is prevented by extending the jack as the new foundation deflects under load. After the new foundation has stopped settling, the jack is replaced by steel wedges which are then encased in concrete. This is known as the Pretest method of underpinning. This method, developed by Lazarus White in constructing subways in New York, has made it possible to build beneath the largest structures without damaging them and to preserve monumental structures.

Out of the two methods of underpinning as explained in the previous paragraphs, the pit method is preferred in case of expansive soils. This technique is effective because the driven piles rest on a firm soil stratum down at greater depth. Even then if some expansion takes place, it is resisted by the jacks provided at the top of each pile.

7.3 Slabs on Expansive Soils

When floor slabs are to be placed on expansive soils, they should generally be laid independent of grade beams carrying walls or partitions so that they are free to move vertically without damaging other parts of the building. Such type of slabs are known as floating slabs. Slab-on-ground is separated from the grade beams by the use of asphalt expansion joints. In basement construction, when the backfill exerts pressure on the grade beam, the expansion joint is under compression and part of the uplift pressure beneath the slab is then transmitted to the grade beam. For effective use of floating

slabs on expansive soils, the grade-beam is extended inside to act as support for the slab. About 3 ft. expansive soil is excavated below the slab and filled with some non-expansive soil like clayey silt. A gap of about 5 to 10 cm. is left between the slab and the ground so that the slab and thereby other structural members are unaffected by expansion [18].

If the floor slab rests on fill between grade beams, especially where the floor is above the outside grade, the fill under the slab should be cohesionless material and not compacted clay. If compacted clay is used for the fill, it will expand laterally as it gains moisture and exert heavy pressure against the grade beams leaving a wide crack between the wall and the slab. An improved slab-on-grade construction method is the installation of a lubricated slip joint between the grade beam and the slab as shown in Fig. 7.2. This installation involves the use of two 1/8-inch masonite strips with silicon lubricant between them. This type of joint system is not affected by lateral pressure thereby allowing free slab movement.

Another form of slab-on-ground construction is the honeycomb system, Fig. 7.3. The development of this kind of system was based on the assumption that comparatively slight movements of some clays reduce or relieve swelling pressures (Means, 1959). The foundation consists of longitudinally split sonotubes placed with the openings down towards the soil. The 2-inch gap between the tubes is filled with sand. The sonotube forms stand up after several days of soak-

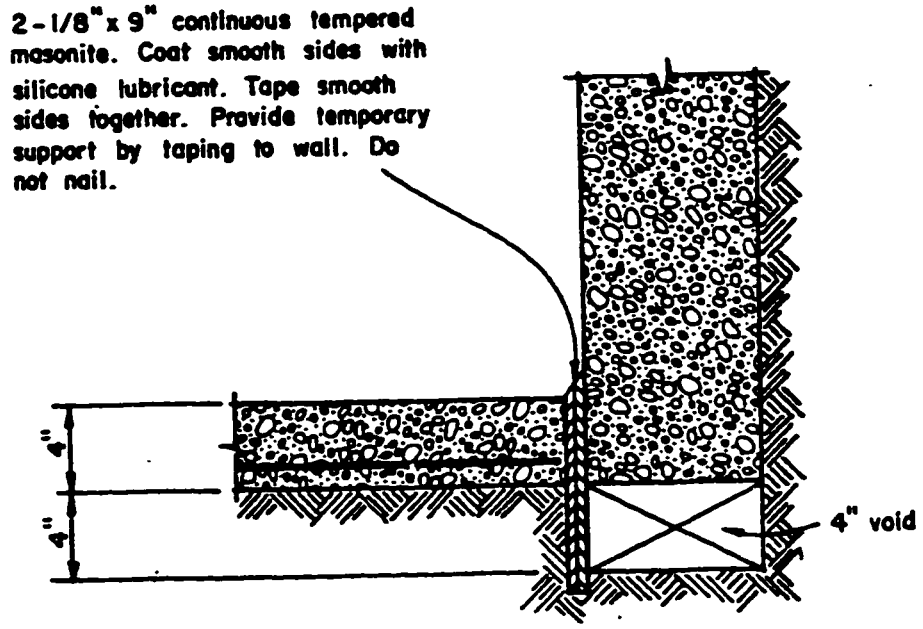


Fig. 7.2 : Floating Slab Construction (Ref. 7)

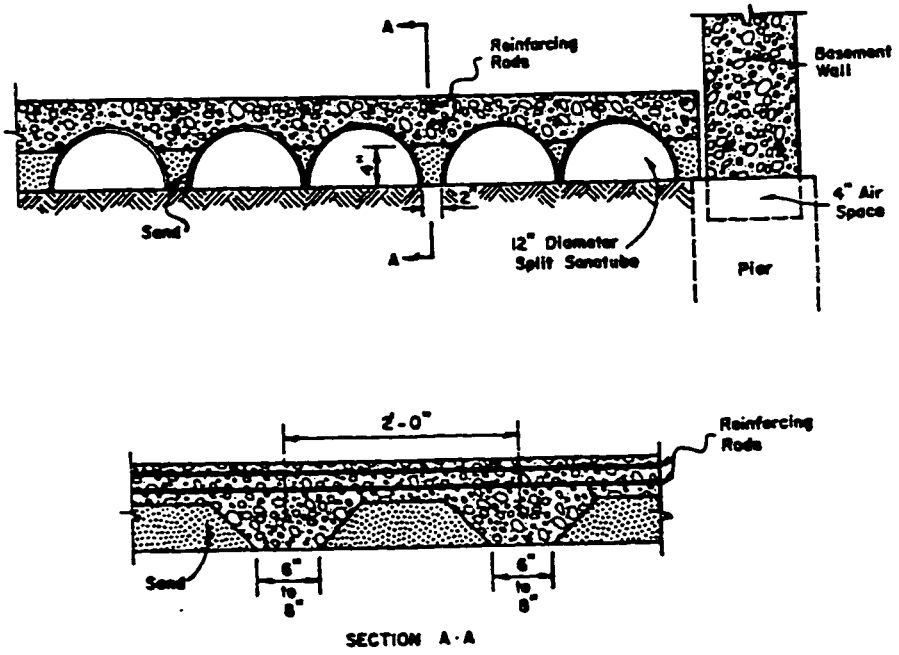


Fig. 7.3 : Honeycomb System of Slab Construction (Ref. 7)

ing. The sand after being released runs out from under the joists. Small reinforcing rods are driven into the ground between forms, which serve to hold the forms in place and to support the steel in the joists. It is believed that as the clay swells, it could expand into these openings and reduce the swelling pressure [2].

7.4 Special Foundation Designs

From consideration of the cause of shrinking and swelling of clays and the condition of surface clays exposed to variable climatic conditions, three methods of preventing damage to buildings are suggested [18].

a. Continuous spread footings are provided by replacing about 3 ft. of the expansive soil by non-expansive low permeability soil.
b. A mat or platform of sufficient stiffness and strength to support the building load without excessive deflection for any possible distribution of supporting pressure under the slab, is laid on the surface of expansive soil and the building constructed on this rigid slab.
c. The building may be supported on footings at a depth below the surface not affected by climatic changes.

The above three methods of foundation designs are discussed in detail as follows.

7.4.1 Continuous Spread Footings

The most common foundation for lightly loaded structures is the continuous footing. However, it should be noted that continuous spread footings cannot be expected to function well in highly expansive soil areas. The use of this kind of construction should be limited to soils with a low degree of expansion; those having a swelling potential of 1 percent and a swelling pressure of less than 3,000 psf ($\approx$ 1.5 Kg/cm^2) [7]. Local building codes sometimes specify the minimum allowable width of footing as 20 inches which is not applicable for footings which are to be placed on expansive soils. To concentrate sufficient dead load pressure on expansive soils, the width of footing should be as narrow as possible.

If a continuous wall footing is to be constructed on a fill, care should be taken in selecting the depth of fill and it should be at least 3 ft beneath the bottom of footings and slabs. Fig. 7.4 shows pressure envelopes illustrating the influence of high contact pressure on swelling. This shows that reducing the size of footing to increase the bearing pressure to exceed the estimated swelling pressure of soil is not always successful. The main reason for this is that the stress increase produced by the reduction in size of footing rapidly decreases with depth, whereas the swelling pressure may not.

For success in placing footings and slabs on selected fills, following precautions should be observed [2]:

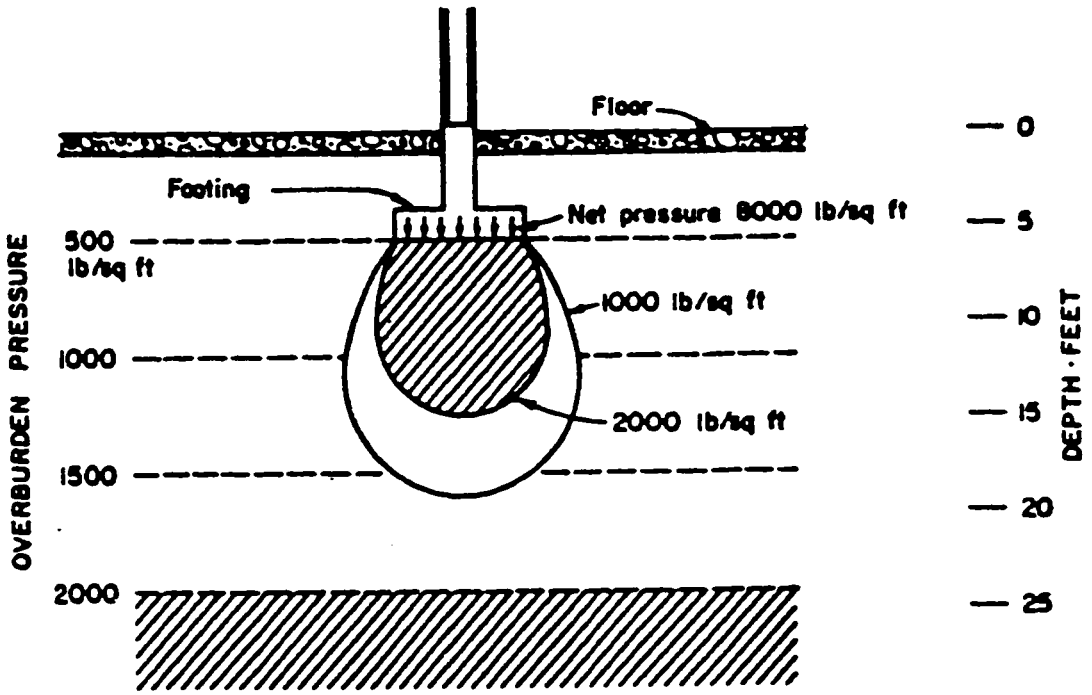


Fig. 7.4 : Continuous Wall Footing on Fill (Ref. 7)

1. There should be at least 3 ft. of selected fill beneath the bottom of footings and slabs.
2. The fill should extend beyond the building line for a distance of at least 10 ft. in every direction [2].
3. The fill should consist of non-expansive soil, preferably mixture of impervious and granular materials like clayey silts.
4. The fill should be compacted to at least 90% of maximum dry density for supporting slabs and 100% maximum dry density for supporting footings as determined by Standard Compaction Test.

7.4.2 Mat Foundations

Mat foundations, sometimes referred to as structural slab-on-ground or reinforced and stiffened slabs, are considered to be both a load supporting as well as a separating element. The slab receives and transmits all the structural load to the underslab soils. The slab should be designed to resist both the positive and the negative moment. Positive moment includes that induced by both dead and live load pressure exerted on the slab. Negative moment consists mainly of those pressures caused by the swelling of the underslab soils. Since the swelling pressure in an expansive soil area can reach many thousand pounds per square foot, negative moment consideration generally controls the design of the mat foundation. If all structural elements are to be placed on a stiffened slab, then slab movement will not affect the stability of the structure. However,

there could be tilting of the mat, but the performance of the building would not be structurally affected. So, there are limitations on the use of such a system as follows [18]:

1. The success of such system so far is limited to moderate swelling soil areas.
2. Configuration of building must be relatively simple.
3. The load exerted on the foundation must be light. Past performance has been limited to residential construction.
4. Single level construction is required. It would be difficult to apply such construction to basement houses with an attached garage.

7.4.3 Deep Foundations

Clay at a considerable depth below the surface does not swell and shrink as much as clay near the surface. The clay at depth, being closer to the water table and farther from the source of evaporation at the surface, is less affected by climatic changes than surface soils. Also expansion of the clay because of increase in water content, is inhibited by the overburden pressure.

Damage from uplift of structures can be reduced by providing deep foundations such as drilled pier or drilled caisson. The drilled pier foundation is used to transfer the structural load from the upper unstable soil to the lower stable soil. The use of this kind of foundation covers a wide range of possibilities; some of them are [18]:

1. Piers drilled into hard bedrock for supporting high column

load.

2. Friction piers bottomed on stiff clays for supporting light structures.
3. Belled piers bottomed on sand and gravel for supporting medium column load, and
4. Long, small-diameter piers drilled into a zone unaffected by moisture change in swelling soil areas.

Piers drilled into materials other than bedrocks are often enlarged at the bottom of the hole for the purpose of increasing the bearing area, and providing resistance against uplift. Such piers are commonly referred to as belled piers or underreamed piers. The ideal bell is in the shape of a frustrum with a vertical side at the bottom. The vertical side is 6 to 12 inches high. The sloping side of the bell should be at an angle at least 60 degrees with the horizontal. Most drillers are capable of providing bells with diameters equal to three times the diameter of the shaft [21].

Fig. 7.5 shows a typical belled pier. The approach has been devised to break the bond between the pier and the clay. The concept was initiated by (Woodward, 1972) in San Antonio, Texas, where highly expansive clays are found. A pipe is introduced into the pier. The concrete filled pipe is designed to carry the column load at the top. The outside of the pipe from the top down to the bottom is coated with a bituminous material. When the pier is lifted by swelling soils, the annulus of the concrete outside the coated section

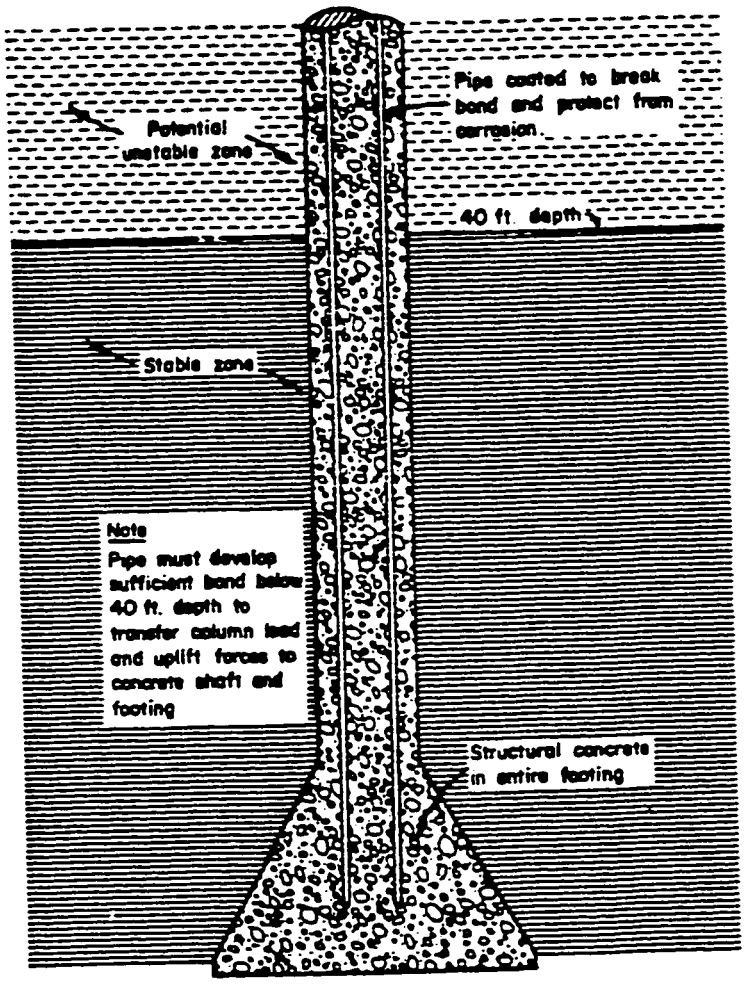


Fig. 7.5 : Typical Bell Pier Design (Ref. 7)

breaks in tension, the bituminous coating breaks the bond, and the upward force is balanced by the viscosity of the bituminous materials [32].

Relatively short piles can be provided for foundations of buildings projected on soils of high and heterogeneous expansive properties as in Al-Qatif area. Such a soil formation has had injurious effects on the constructions due to progressive deformation in some structural elements causing cracks, fissures and heave especially in light structures (Leopoldo, 1980).

The most economical way of constructing these short piles is with pile drilling equipment. The diameter of the base is kept twice the shaft diameter as in the case of belled piers. The pile length is limited to 20 ft. with diameter of 3 ft. The pile capacity is determined according to following equations [18]:

For cohesive soils ($\varphi = 0$),

$$q_d = cN_c + \gamma DN_q \quad \text{(Skempton, 1951)}$$

For soils with cohesion and friction,

$$q_d = 1.3\ cN_c + \gamma DN_q + 0.6\ \gamma BN_\gamma \quad \text{(Meyerhof, 1955)}$$

where,

q_d = pile capacity (psf)

c = cohesion (psf)

D = length of pile (ft)

B = shaft diameter (ft)

γ = unit weight of soil (pcf)

N_c, N_q, N_γ = bearing capacity parameters

To isolate the upper part of the pile from the surrounding soil a light metallic form 1/16 inches (approximately 1 1/2) thick is used. It is further specified that the diameter of the shaft't upper part should be equal to 3/4 the nominal pile diameter (B), and that a concrete cap be constructed to avoid entrance of surface water to the soil-pile free annular space. The grade beams and floors are separated from the soil by a height of not less than 12 inches (30 cms). The final design of such a short pile is shown in Fig. 7.6.

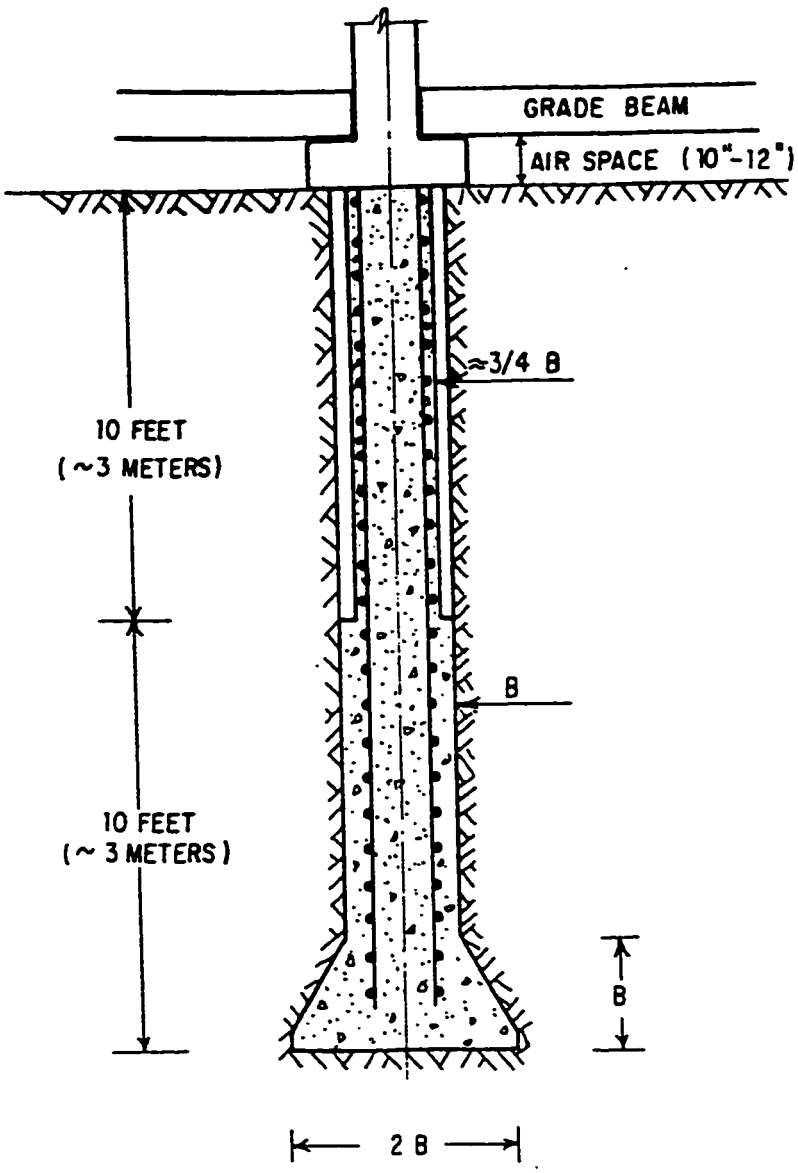


Fig. 7.6 : Final Design of a Short Pile
(Leopodo, 1980)

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

From the geotechnical investigation and mineralogical identification of Al-Qatif expansive clays, the following conclusions can be drawn.

1. Geotechnical properties of Al-Qatif soil show high swelling potential with variation in properties from one location to another over small area.
2. The two types of direct swelling tests show high values of swelling pressure and % swell.
3. Al-Qatif soils based on the swelling characteristics can be characterized as highly expansive with swelling pressure values as high as 3.78 Kg/cm^2 and % swell of 14%.
4. The recognition of layering system (horizontal or vertical) is very important before conducting any test otherwise the results may be erratic and erroneous.
5. Chemical analysis showed the presence of relatively low amount of organic matter. No fossils were detected. The probable source of this organic matter was found to be plant debris and sea weeds. All soil samples were alkaline in character with pH values around 7.9 possessing high cec values.

6. X-ray diffraction and SEM analyses of soil samples confirm the presence of montmorillonite - the most expanding clay mineral. Among clay minerals, polygorskite was dominating followed by montmorillonite. Illite and kaolinite were absent.. Among non-clay minerals, quartz was common while dolomite was detected in soil samples from one of the blocks only.
7. Fabric study indicates that Al-Qatif clays resemble with the most common fabric of montmorillonite. Edge to edge and edge to face (flocculated) arrangement of particles was observed along layers while SEM microphotographs perpendicular to layers show face to face attachment with clear layers and large voids. Flakes morphology (montmorillonite) was common with some needle fibers confirming presence of polygorskite.
8. Soil samples compacted wet of optimum were found to swell less than soil samples compacted dry of optimum at the same dry density.
9. The two techniques used for chemical stabilization were proved to be successful in reducing soil expansion.
10. Chemical stabilization with various kinds of compounds indicated that 4-8% of commercial lime is preferred for preconstruction treatment of Al-Qatif clays due to its low cost and comparatively better results.
11. Curing with commercial lime for 7 and 14 days indicates that soil-lime reaction is more pronounced during first week of lime treatment. So, for proper stabilization the site should be kept

untouched for one or two weeks.

8.2 Recommendations for Further Research

1. A thorough geotechnical investigation may be carried out throughout the Kingdom to locate the remaining areas possessing expansive soils and a complete map of expansive formations is to be prepared.
2. Laboratory simulation tests for soil-structure interaction may be performed for correlation and calibration of geotechnical properties obtained from the site investigation.
3. A study of the interaction of expansive soil with substructures (e.g. foundations) in the field be carried out and correlations can be made with laboratory results.
4. Field treatments of expansive soils with different reagents may be studied to suggest the most appropriate and economical method to combat swelling and differential settlement problem.
5. Statistical analysis of laboratory and field data is necessary to develop empirical relationships between different parameters related to expansive soils.

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APPENDIX

DATA FOR SWELL PRESSURE TEST WITH DIFFERENT PERCENTAGES OF POTASSIUM CHLORIDE

FILE : SPKCL

(Ref.: Fig. 6.11)

COMPUTER OUTPUT

TIME IN HOURS AND SWELL PRESSURE IN KG/SQ.CM.

FILE : SPKCL

(Ref.: Fig. 6.11)

Time and Swell Pressure Data for Soil Sample from Block # 03 Treated with Potassium Chloride

Time	Swell Pressure	Time	Swell Pressure	Time	Swell Pressure
1 143 3		2.0500	2.9494	28.5500	3.8184
143		2.3000	3.0743	29.5500	3.8083
0.0014	0.0536	2.5500	3.1857	30.5500	3.7883
0.0028	0.0635	2.8000	3.2729	31.5500	3.7914
0.0042	0.0734	3.0500	3.3545	32.5500	3.7914
0.0056	0.0794	3.3000	3.4141	33.5500	3.7904
0.0070	0.0794	3.5500	3.4658	34.5500	3.7904
0.0084	0.0773	3.8000	3.5173	35.5500	3.7893
0.0098	0.0794	4.0500	3.5510	36.5500	3.7875
0.0112	0.0813	4.3000	3.5889	38.5500	3.7886
0.0126	0.0833	4.5500	3.6127	40.5500	3.7885
0.0140	0.0852	4.8000	3.6425	42.5500	3.7884
0.0154	0.0833	5.0500	3.6563	44.5500	3.7885
0.0168	0.0852	5.3000	3.6882	46.5500	3.7885
0.0208	0.0852	5.5500	3.6900	48.5500	3.7837
0.0250	0.0873	5.8000	3.7120	50.5500	3.7875
0.0291	0.0912	6.0500	3.7159	52.5500	3.7884
0.0333	0.1011	6.3000	3.7439	54.5500	3.7883
0.0375	0.1150	6.5500	3.7379	56.5500	3.7885
0.0416	0.1309	7.0500	3.7517	58.5500	3.7886
0.0458	0.1448	7.5500	3.7637	60.5500	3.7886
0.0500	0.1547	8.0500	3.7776	62.5500	3.7885
0.0666	0.2064	8.5500	3.7836	70.5500	3.7888
0.0833	0.2522	9.0500	3.7914	80.5500	3.7888
0.1000	0.2979	9.5500	3.8014	90.5500	3.7881
0.1166	0.3395	10.0500	3.8053	100.550	3.7881
0.1333	0.3813	10.5500	3.8074	110.550	3.7884
0.1500	0.4586	11.0500	3.8092	120.550	3.7881
0.1666	0.4586	11.5500	3.8113	130.550	3.7788
0.1888	0.4965	12.0500	3.8113	140.550	3.7898
0.2000	0.5321	12.5500	3.8113	150.550	3.7890
0.2166	0.5699	13.0500	3.8113	160.550	3.7888
0.2500	0.6455	13.5500	3.8113	170.550	3.7890
0.2800	0.7189	14.0500	3.8113	180.550	3.7890
0.3100	0.7923	14.5500	3.8113	190.550	3.7889
0.3500	0.8619	15.0500	3.8134	143	
0.3833	0.9314	15.5500	3.8113	0.0014	0.0536
0.4166	0.9988	16.0500	3.8092	0.0028	0.0635
0.4500	1.0644	16.5500	3.8134	0.0042	0.0734
0.4833	1.1321	17.0500	3.8152	0.0056	0.0794
0.5166	1.1956	17.5500	3.8173	0.0070	0.0794
0.5500	1.2591	18.0500	3.8152	0.0084	0.0773
0.6330	1.4120	18.5500	3.8173	0.0098	0.0794
0.7166	1.5591	19.0500	3.8272	0.0112	0.0813
0.8000	1.6940	19.5500	3.8272	0.0126	0.0833
0.8833	1.8210	20.0500	3.8262	0.0140	0.0852
0.9666	1.9404	20.5500	3.8251	0.0154	0.0833
1.0500	2.0496	21.0500	3.8273	0.0168	0.0852
1.1333	2.1607	21.5500	3.8283	0.0208	0.0852
1.2166	2.2621	22.5500	3.8174	0.0250	0.0873
1.3000	2.3554	23.5500	3.8084	0.0291	0.0912
1.3833	2.4369	24.5500	3.8172	0.0333	0.1011
1.4666	2.5143	25.5500	3.8182	0.0375	0.1150
1.5500	2.5898	26.5500	3.8272	0.0416	0.1309
1.8000	2.7863	27.5500	3.8073	0.0458	0.1448

Contd.

Time	Swell Pressure	Time	Swell Pressure	Time	Swell Pressure
0.0500	0.1547	8.0500	3.7776	62.5500	3.7555
0.0666	0.2064	8.5500	3.7836	70.5500	3.7418
0.0833	0.2522	9.0500	3.7914	80.5500	3.7318
0.1000	0.2979	9.5500	3.8014	90.5500	3.7141
0.1166	0.3395	10.0500	3.8053	100.550	3.7081
0.1333	0.3813	10.5500	3.8074	110.550	3.6744
0.1500	0.4586	11.0500	3.8092	120.550	3.6683
0.1666	0.4586	11.5500	3.8113	130.550	3.6386
0.1888	0.4965	12.0500	3.8113	140.550	3.6007
0.2000	0.5321	12.5500	3.8113	150.550	3.6148
0.2166	0.5699	13.0500	3.8113	160.550	3.5769
0.2500	0.6455	13.5500	3.8113	170.550	3.5769
0.2800	0.7189	14.0500	3.8113	180.550	3.5432
0.3100	0.7923	14.5500	3.8113	190.550	3.5293
0.3500	0.8619	15.0500	3.8134	143	
0.3833	0.9314	15.5500	3.8113	0.0014	0.0750
0.4166	0.9988	16.0500	3.8092	0.0028	0.0768
0.4500	1.0644	16.5500	3.8134	0.0042	0.0844
0.4833	1.1321	17.0500	3.8152	0.0056	0.0995
0.5166	1.1956	17.5500	3.8173	0.0070	0.1111
0.5500	1.2591	18.0500	3.8152	0.0084	0.1218
0.6330	1.4120	18.5500	3.8173	0.0098	0.1314
0.7166	1.5591	19.0500	3.8272	0.0112	0.1401
0.8000	1.6940	19.5500	3.8212	0.0126	0.1479
0.8833	1.8210	20.0500	3.8212	0.0140	0.1547
0.9666	1.9404	20.5500	3.8251	0.0154	0.1620
1.0500	2.0496	21.0500	3.8213	0.0168	0.1688
1.1333	2.1607	21.5500	3.8233	0.0208	0.1887
1.2166	2.2621	22.5500	3.8134	0.0250	0.2072
1.3000	2.3554	23.5500	3.8074	0.0291	0.2250
1.3833	2.4369	24.5500	3.8152	0.0333	0.2430
1.4666	2.5143	25.5500	3.8152	0.0375	0.2605
1.5500	2.5898	26.5500	3.8272	0.0416	0.2775
1.8000	2.7863	27.5500	3.8053	0.0458	0.2929
2.0500	2.9494	28.5500	3.8134	0.0500	0.3081
2.3000	3.0743	29.5500	3.8073	0.0666	0.3598
2.5500	3.1857	30.5500	3.7893	0.0833	0.4029
2.8000	3.2729	31.5500	3.7954	0.1000	0.4450
3.0500	3.3545	32.5500	3.7954	0.1166	0.4855
3.3000	3.4141	33.5500	3.7954	0.1333	0.5234
3.5500	3.4658	34.5500	3.7914	0.1500	0.5595
3.8000	3.5173	35.5500	3.7893	0.1666	0.5950
4.0500	3.5510	36.5500	3.7875	0.1888	0.6280
4.3000	3.5889	38.5500	3.7836	0.2000	0.6601
4.5500	3.6127	40.5500	3.7755	0.2166	0.6910
4.8000	3.6425	42.5500	3.7794	0.2500	0.7498
5.0500	3.6563	44.5500	3.7815	0.2800	0.8054
5.3000	3.6882	46.5500	3.7655	0.3100	0.8574
5.5500	3.6900	48.5500	3.7737	0.3500	0.9063
5.8000	3.7120	50.5500	3.7875	0.3833	0.9533
6.0500	3.7159	52.5500	3.7914	0.4166	0.9972
6.3000	3.7439	54.5500	3.7893	0.4500	1.0398
6.5500	3.7379	56.5500	3.7655	0.4833	1.0811
7.0500	3.7517	58.5500	3.7596	0.5166	1.1208
7.5500	3.7637	60.5500	3.7556	0.5500	1.1590

Contd.

Time	Swell Pressure	Time	Swell Pressure
0.6330	1.2499	18.5500	2.5044
0.7166	1.3331	19.0500	2.5025
0.8000	1.4091	19.5500	2.4944
0.8833	1.4799	20.0500	2.4842
0.9666	1.5455	20.5500	2.4861
1.0500	1.6061	21.0500	2.4853
1.1333	1.6657	21.5500	2.4777
1.2166	1.7211	22.5500	2.4628
1.3000	1.7700	23.5500	2.4508
1.3833	1.8140	24.5500	2.4424
1.4666	1.8560	25.5500	2.4317
1.5500	1.8970	26.5500	2.4262
1.8000	2.0049	27.5500	2.4058
2.0500	2.0969	28.5500	2.3990
2.3000	2.1680	29.5500	2.3857
2.5500	2.2331	30.5500	2.3692
2.8000	2.2848	31.5500	2.3635
3.0500	2.3316	32.5500	2.3535
3.3000	2.3661	33.5500	2.3426
3.5500	2.3990	34.5500	2.3316
3.8000	2.4273	35.5500	2.3204
4.0500	2.4471	36.5500	2.3102
4.3000	2.4686	38.5500	2.2882
4.5500	2.4814	40.5500	2.2618
4.8000	2.4981	42.5500	2.2543
5.0500	2.5064	44.5500	2.2378
5.3000	2.5227	46.5500	2.2156
5.5500	2.5247	48.5500	2.1947
5.8000	2.5362	50.5500	2.1887
6.0500	2.5381	52.5500	2.1772
6.3000	2.5509	54.5500	2.1583
6.5500	2.5477	56.5500	2.1382
7.0500	2.5537	58.5500	2.1231
7.5500	2.5579	60.5500	2.1100
8.0500	2.5616	62.5500	2.1101
8.5500	2.5629	70.5500	2.0575
9.0500	2.5629	80.5500	1.9968
9.5500	2.5603	90.5500	1.9613
10.0500	2.5587	100.550	1.9255
10.5500	2.5563	110.550	1.8839
11.0500	2.5532	120.550	1.8664
11.5500	2.5496	130.550	1.8390
12.0500	2.5462	140.550	1.8076
12.5500	2.5425	150.550	1.8116
13.0500	2.5394	160.550	1.7940
13.5500	2.5355	170.550	1.7938
14.0500	2.5323	180.550	1.7802
14.5500	2.5287	190.550	1.7765
15.0500	2.5258		
15.5500	2.5224		
16.0500	2.5179		
16.5500	2.5164		
17.0500	2.5143		
17.5500	2.5125		
18.0500	2.5083		